JUNE 1955

Journal

VOL. 47 • NO. 6

AMERICAN WATER WORKS ASSOCIATION

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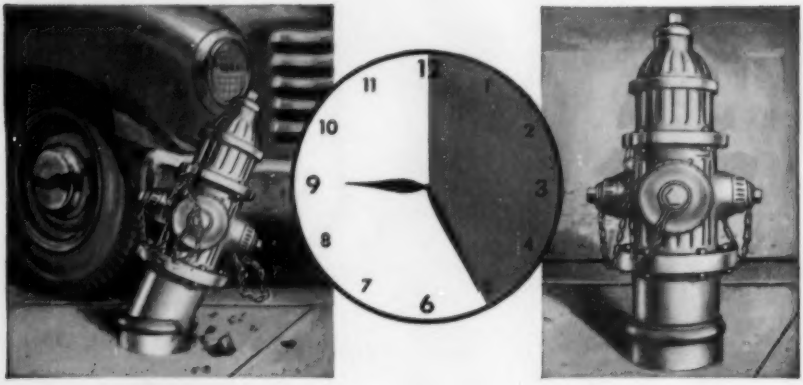

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June 1955

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Journal AWWA is published monthly at Prince & Lemon Sts., Lancaster, Pa., by the Am. Water Works Assn., Inc., 521 Fifth Ave., New York 17, N.Y., and entered as second class matter Jan. 23, 1943, at the Post Office at Lancaster, Pa., under the act of Aug. 24, 1912. Accepted for mailing at a special rate of postage provided for in paragraph (d-2), Section 34.40, P. L. & R. of 1948. Authorized Aug. 6, 1918. $7.00 of members' dues are applied as a subscription to the JOURNAL; additional single copies to members—60 cents; single copies to non-members—85 cents. Indexed annually in December; and regularly by *Industrial Arts Index* and *Engineering Index*. Microfilm edition (for JOURNAL subscribers only) by University Microfilms, Ann Arbor, Mich.

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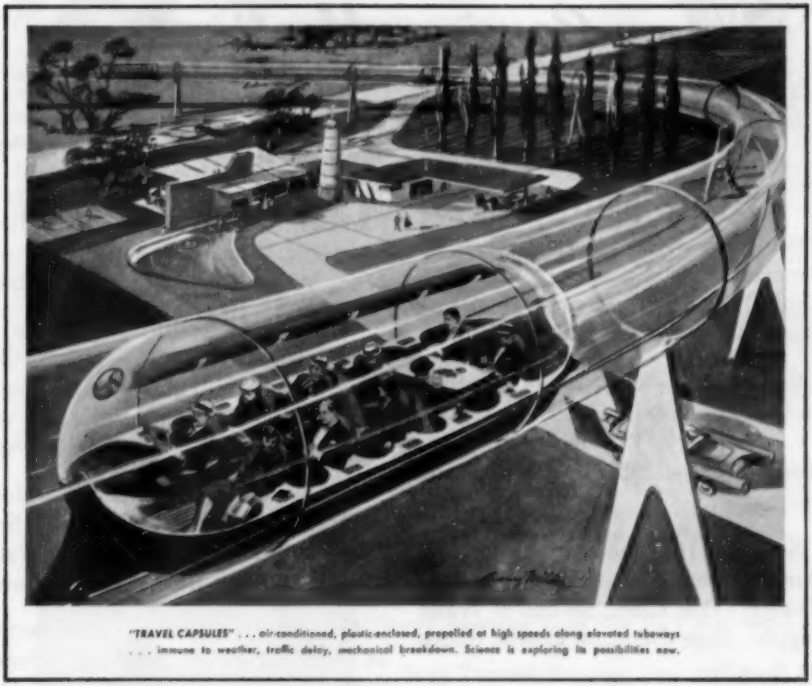

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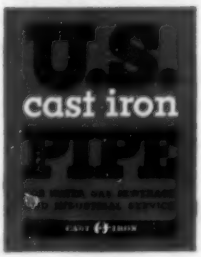

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Sep. 12–14—Kentucky-Tennessee Section at Phoenix Hotel, Lexington, Ky. Secretary, J. Wiley Finney Jr., Howard K. Bell, 553 S. Limestone St., Lexington.

Sep. 14–16—Michigan Section at Durant Hotel, Flint. Secretary, T. L. Vander Velde, Chief, Section of Water Supply, State Dept. of Health, Lansing 4.

Sep. 19–21—Rocky Mountain Section at Connor Hotel, Laramie, Wyo. Secretary, J. W. Davis, Dist. Mgr., Transite Pipe Div., Johns-Manville Sales, Inc., Denver, Colo.

Sep. 21–23—Ohio Section at Neil House, Columbus. Secretary, M. E. Druley, Dist. Mgr., Dayton Power & Light Co., Wilmington.

Sep. 21–23—Wisconsin Section at Hotel Schroeder, Milwaukee. Secretary, Leon A. Smith, Supt., Water & Sewerage, City Hall, Madison 3.

Sep. 25–27—Missouri Section at the Connor Hotel, Joplin, Mo. Secretary, W. A. Kramer, Room 3, 6th Floor State Office Bldg., Jefferson City, Mo.

Oct. 5–7—North Central Section at Radisson Hotel, Minneapolis, Minn. Secretary, Leonard N. Thompson, Gen. Mgr., Water Dept., St. Paul 2, Minn.

Oct. 16–19—Southwest Section at Gunter Hotel, San Antonio, Tex. Secretary, Leslie A. Jackson, Mgr.-Engr., Munic. Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

Oct. 19–21—Iowa Section at Hotel Fort Des Moines, Des Moines. Secretary, H. V. Pedersen, Supt., Water Works, Munic. Bldg., Marshalltown.

Oct. 20–21—West Virginia Section at Waldo Hotel, Clarksburg. Secretary, Harry K. Gidley, Director, Div. of San. Eng., State Health Dept., Charlestown.

Oct. 20–22—New Jersey Section at Hotel Madison, Atlantic City. Secretary, C. B. Tygert, Wallace & Tiernan Inc., Box 178, Newark 1.

OTHER ORGANIZATIONS

Jun. 26–Jul. 1—American Society for Testing Materials, Atlantic City, N.J.

Jul. 18–23—International Water Supply Assn. Congress, London.

Sep. 18–22—New England Water Works Assn., Lake Placid Club, Lake Placid, N.Y.

Oct. 10–13—Federation of Sewage & Industrial Wastes Assns., Ambassador Hotel, Atlantic City, N.J.

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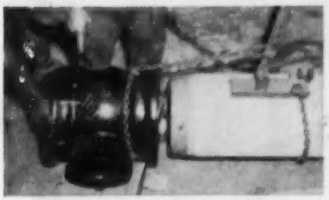

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This is another example of the fast, low-cost, maintenance-free jobs you can do with Dresser-Coupled steel pipe. Providing virtually everlasting high-capacity lines, this is truly the modern way to *deliver water cheaper.*

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Installed by the Arthur Pew Construction Company of Atlanta, this 26,990-foot Dresser-Coupled 36-inch steel line extends from the new Tuscaloosa Reservoir on Yellow Creek to the pumping station. Not a single leak was found on a line test of 15,000 feet of pipe.

√Wherever water flows, steel pipes it best. Always put steel pipe and Dresser Couplings in your specifications.

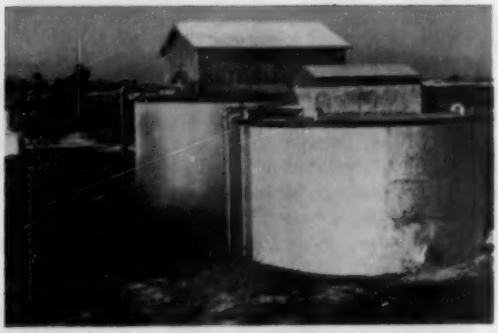
DRESSER®

Dresser Manufacturing Division, 69 Fisher Ave., Bradford, Pa. (One of the Dresser Industries). Warehouses: 1121 Rothwell St., Houston; 101 S. Airport Blvd., S. San Francisco. Sales Offices also in: New York, Philadelphia, Chicago, Toronto.

HOLLYWOOD (Florida) ...regenerating its

1

Pumps draw sea water from wells beneath this station. Being inland, these wells deliver a partially filtered sea water. (Regenerating costs are now $25 less per million gallons than when dry salt was used.)



2

To prevent marine growths which would foul equipment, the sea water is aerated and chlorinated.

3

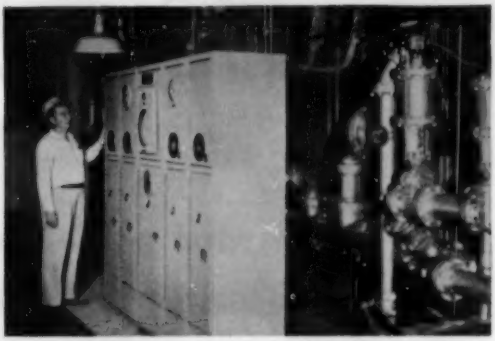
Sea water then passes through Permutit pressure filters (left) and is ready for regenerating the Permutit softeners (right). Sea water pumpage is kept to a minimum because high-capacity Permutit Q, used in the softeners, requires less salt for regeneration.

WATER CONDITIONING HEADQUARTERS

SAVES $32,980 A YEAR
softeners with sea water

4

City Engineer George A. Gieseke inspects the Permutit control panel. It backwashes, regenerates and rinses the 4 newest softeners and places them on the line . . . *all without aid from the plant operator!* Two men per shift operate the entire water plant.



5

Hollywood is growing fast. Their first four Permutit softeners went on the line twenty years ago. Now there are 12 . . . and plant space for more. "We're in good shape," says City Engineer Gieseke.

It's easy to see why Consulting Engineers *Reynolds, Smith & Hills* chose Permutit equipment . . . for its long service life and low operating costs. And we'll be glad to help you make sure your water supply is "in good shape" for the future. Write to: The Permutit Company, Dept. JA-6, 330 West 42nd Street, New York 36, N. Y.

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will be no problem
if you call in National now!

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Call in National today!

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TRINITY VALLEY IRON AND STEEL CO.

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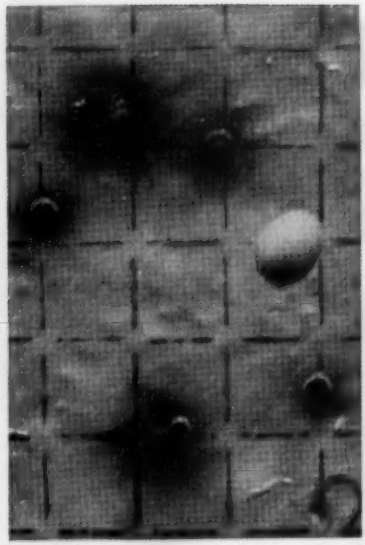

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•Typhoid Isolation

S. typhosa colonies cultured on MF® surface with Bismuth Sulfite medium showing shiny black centers and colorless periphery. The area surrounding the colony has black metallic sheen. Non-typhoid colonies are green or gray.

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Kabler and Clark* have reported the isolation of S. typhosa on the **MF** with Bismuth Sulfite (Wilson Blair Broth). "Recovery rates from water by the membrane method have been superior to those of other commonly used laboratory procedures."

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A comprehensive summary of literature on many new techniques and procedures with instructions for the use of **MF** membrane filters and accessory equipment is available on request from the laboratories of the

MILLIPORE FILTER CORPORATION

Watertown 72, Mass., U.S.A.

* Kabler, P. W., Clark, H. F.
Amer. J. Public Health 42:390-92 Apr. 1952.
"The Use of Differential Media with the Membrane Filter."

† Collet, Elizabeth, Johnston, R., Ey, Leo F., Croft, C. C.
Amer. J. Public Health 43:1438-40 Nov. 1953.
"Isolation of Salmonella Typhosa from Well Water by the Membrane Filter Technic."

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Because:

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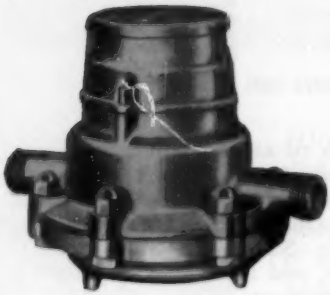
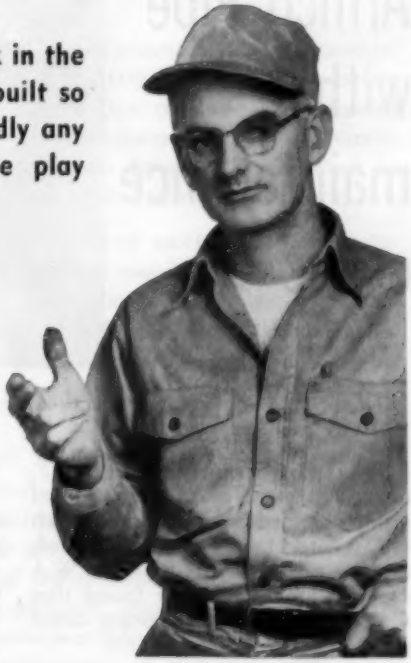
"American Meters reduce work in the meter shop because they are built so well and so simply there's hardly any maintenance. That's why we play safe and buy American."





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Staunton, Virginia, installs Armco Pipe with maintenance crews

Installing Armco Welded Steel Pipe at Staunton, Virginia.

The city of Staunton, Virginia, needed a water line. They ordered Armco Welded Steel Pipe, in 16-inch O.D., with a .188-inch wall; lined, coated and wrapped to AWWA specifications. And they found that their city maintenance group could make the entire installation.

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Armco Welded Steel Pipe is supplied in uniform lengths up to 50 feet, instead of the usual random or double random lengths. This means there are fewer sections to handle and fewer joints to make. Work goes fast.

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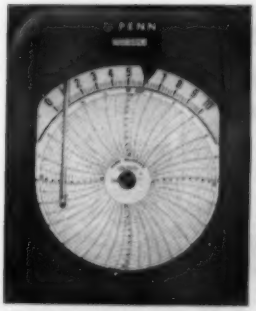
Outside coatings are supplied to AWWA specifications, and a spun enamel lining will prevent tuberculation. Write us for more data. Armco Drainage & Metal Products, Inc., 5505 Curtis Street, Middletown, Ohio. Subsidiary of Armco Steel Corporation. In Canada: write Guelph, Ontario.

WHEREVER WATER FLOWS, STEEL PIPES IT BEST

ARMCO Welded Steel Pipe

MEETS A.W.W.A. SPECIFICATIONS

ARMCO



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Loss of Head
Pressure
Temperature
Pump Operation

TELEPHONE LINE TRANSMISSION

The origin of Penn's present line of instrumentation goes back to 1909 when Cochrane meters were introduced. One of the earliest testing programs for open channel flow measurement was undertaken to develop these meters as reported at the 1914 A.S.M.E. meeting.

In 1918, an improved mechanical flow meter with no stuffing box was developed and in 1935 the first null-balance, servo-powered electric meter was produced. More than 10,000 of these meters have been installed throughout the world.

HIGHEST REPRODUCIBLE ACCURACY distinguishes Penn meters. Such accuracy is accomplished only by a null-balance, servo-powered electric meter. To this unexcelled high fidelity of electrical transmission and recording, Penn adds the additional refinement of a calibration cam. This cam precisely matches the calibration of the recorder-totalizer with that of the differential producer. The result is "matched metering" and higher over-all accuracy.

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PENN *Instruments*

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Morton makes salt for every industrial use under the sun — canning, meat packing, freeze-proofing, water softening, brine making — to mention but a few.

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Industrial Division, Dept. JA-6

120 South LaSalle Street, Chicago 3, Ill. • Phone: Financial 6-1300

Difficult problems overcome—yet water costs only 3½ cents per 1000 gallons!

Chardon, Ohio solves water problems

WITH A

BJ SUBMERSIBLE

Chardon, Ohio, faced these difficult problems:

1. The proposed well site was a flood area.
2. It was on a golf course where unsightly pump houses would mar the landscape.
3. It was 1½ miles from the water tower.

RESULTS TALK!

The Chardon Board of Public Affairs worked with Byron Jackson Co. engineers. A 30 horsepower Subette rated at 250 gpm against a head of 360 ft. was installed in a simply-constructed pit. Unseen and completely submerged, this single Subette pumps one-half of the town's water supply. It eliminated an unwanted pump house and solved the dangers of flood-out during high water. In addition, the BJ Subette is pumping water at an operating cost of only 3½ cents per 1000 gallons!

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Write for complete information

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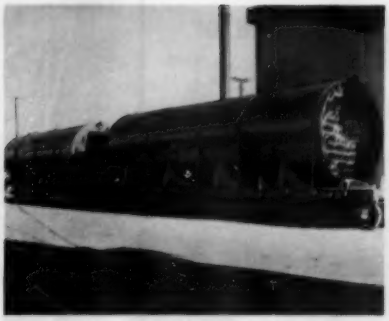
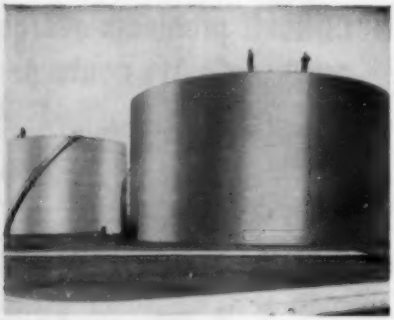
PUMP DIVISION
P.O. BOX 2017, TERMINAL ANNEX
LOS ANGELES 54, CALIFORNIA
Sales offices in all principal cities





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HERSEY WATER METERS

have been approved and recommended by many experienced water works engineers for their unusual reliability and accurate registration.

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IOWA hydrants and valves are never "orphans"!

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Below are additional facts to keep in mind when you plan to expand or replace any part of your city water works.

IOWA HYDRANTS

Provide full, truly unrestricted water way.

Open *with* the pressure for faster water delivery.

Close *against* the pressure to eliminate water hammer, provide flushing action.

Assure positive drainage.

Are available for any type of connection.

All working parts removable through top—no digging or special tools required.

Meet all A.W.W.A. specifications.

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THE LATEST WORD IN WATER TREATMENT

Surgeability is defined as stability of performance under rapidly changing and unpredictable conditions including flow. This characteristic is vitally important in clarification and cold process softening installations. Surgeability is designed and built into the Graver Reactivator®.

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Low Sludge Level . . . providing maximum depth of clarified water between sludge-clear water separation zone and the effluent collector.

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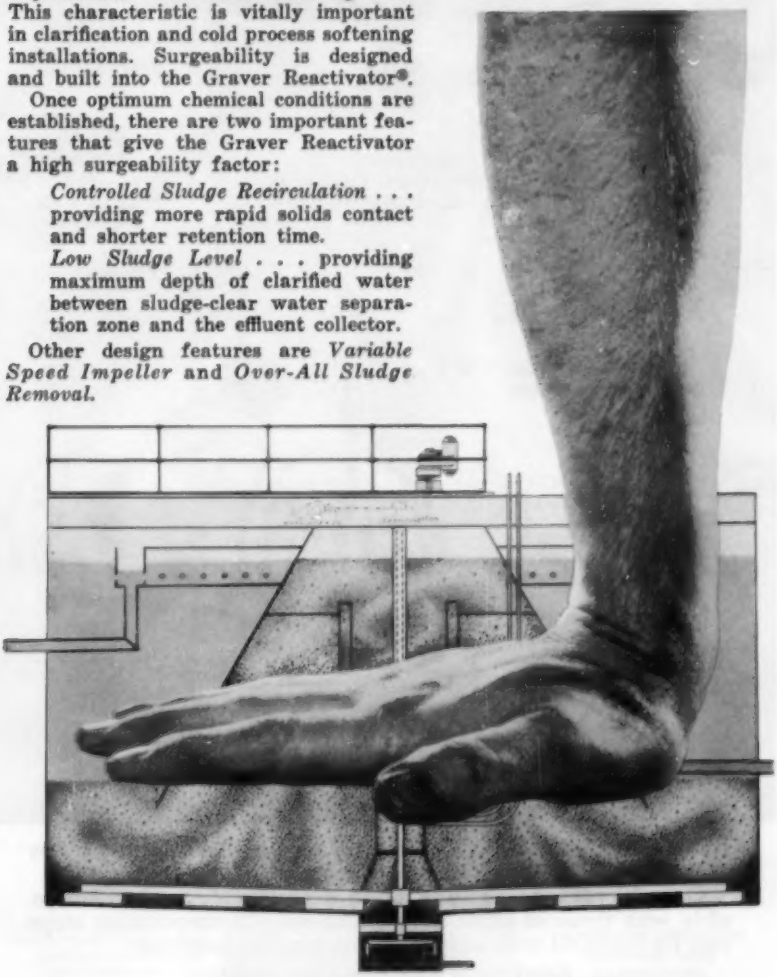

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Department NK

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GRAVER WATER CONDITIONING CO.

A Division of Graver Tank & Mfg. Co., Inc.

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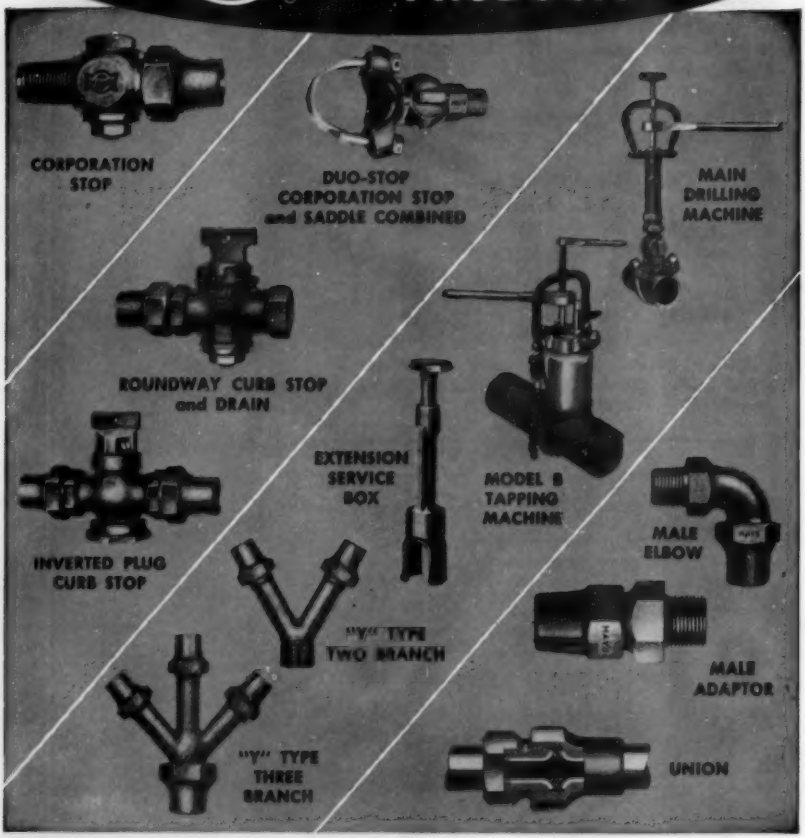

It Pays to Buy Hays . . . The items shown above are only a few of the HAYS complete line of water service products.

Over 80 years of manufacturing experience . . . interchangeable with those of other manufacturers . . . corporation stops can be installed with any standard tapping machine.

Send for HAYS complete catalog.

HAYS

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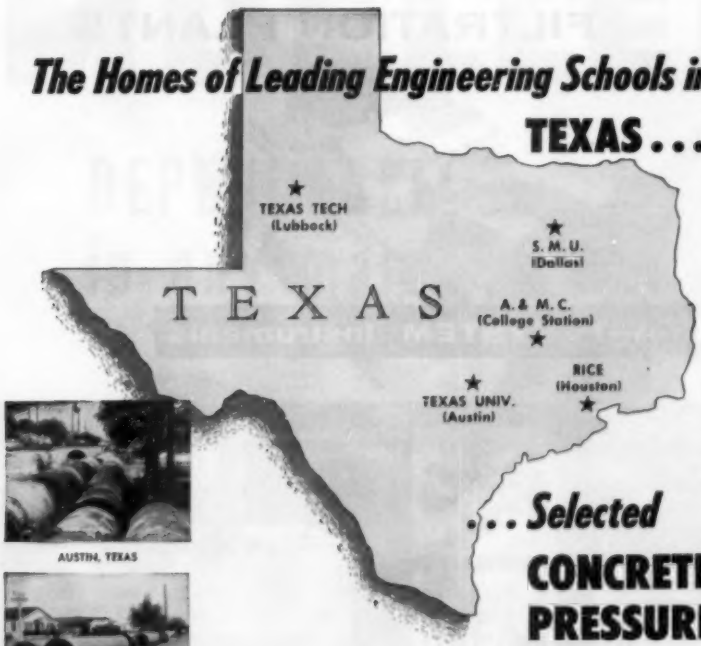
HAYS MANUFACTURING CO.

ERIE, PA.

The Homes of Leading Engineering Schools in TEXAS . . .



AUSTIN, TEXAS

LUBBOCK, TEXAS

COLLEGE STATION, TEXAS

. . . Selected CONCRETE PRESSURE PIPE

The five cities shown on the above map where the leading Engineering Schools of TEXAS are located have large CONCRETE PRESSURE PIPE installations in their water systems. In other words, the Engineers of today selected for the Engineers of tomorrow CONCRETE PRESSURE PIPE through which they receive pure clear sparkling drinking water. Such popularity of Concrete Pressure Pipe in Texas must be deserved.

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Concrete PRESSURE Pipe

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AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION
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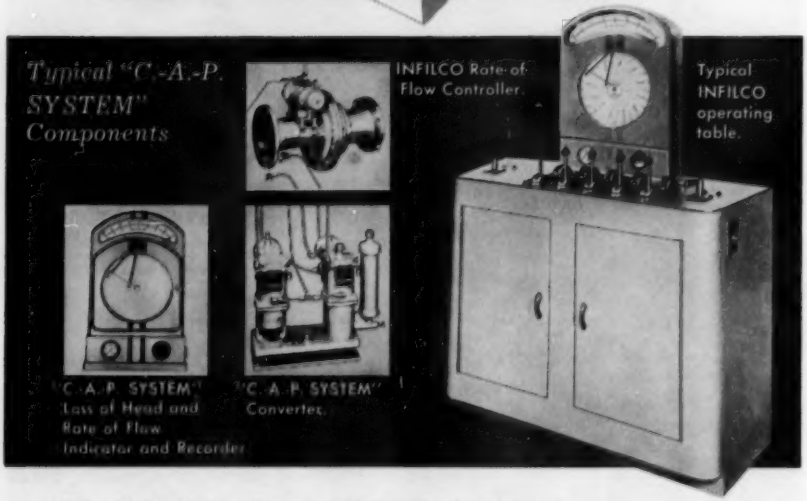
C.-A.-P. SYSTEM® Instruments


Typical "C.-A.-P. SYSTEM" Components
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Typical INFILCO operating table.
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Shipment from stock
Fully Warranted

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Designed for easy insertion under pressure, these stops are made entirely from the finest Water Works Bronze. All keys are precision machined and individually ground and lapped into the stop body. Each stop is carefully inspected and pressure tested. Exposed threads are coated with plastic to prevent shipping damage.

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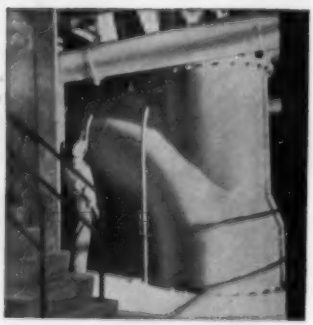
MUELLER CO. DECATUR, ILL.

Dependable since 1857

FACTORIES & OFFICES AT DECATUR · CHATTANOOGA · LOS ANGELES · SARNIA, ONTARIO

OAKLAND CAN REMOVE
648 MILLION GALLONS OF STORM WATER PER DAY
WITH FOUR HUGE WHEELER-ECONOMY PUMPS

Each of the four Wheeler-Economy Vertical Mixed-Flow Pumps in the new Oakland Storm Water Pumping Station has a capacity of 112,250 g. p. m. They are among the largest pumps ever built—54 x 66 in., weighing approximately 20 tons apiece. Average head is 31 ft. with pump inlets nearly 25 ft. below the engine floor. In any development of the magnitude of the Oakland, California storm water project, pump performance must measure up to the millions of dollars invested. Wheeler-Economy has pioneered in the development of large axial and mixed-flow pumping equipment. That's why so many municipalities play it safe with Wheeler-Economy engineering skill and experienced workmanship. (Write for Bulletin on the Oakland story.)



One of huge Wheeler-Economy Pumps at Oakland, California. Conduit sizes draining into pumping station range from 24 to 108 in., large enough for an automobile.

WE-611

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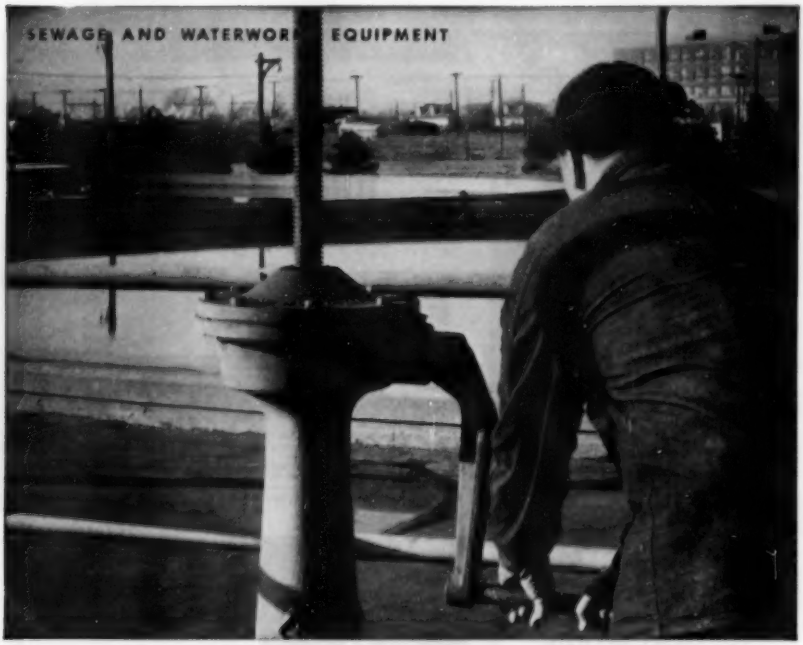

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TURN THEM EASILY—ONCE A DAY OR ONCE A YEAR. THEY RESIST CORROSION.

Rust and efficiency—like oil and water—don't mix. In gate and valve stems rust costs money. You avoid this by specifying stems made of Everdur*—*strong, rustproof and corrosion-resistant.*

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Depending on the type or composition, you can work Everdur Alloys hot or cold, and readily form, forge, weld and machine them. They are available in plates, sheets, rods, tubes, electrical conduit and casting ingots.

Write for free booklet, "Everdur Copper-Silicon Alloys for Sewage and Waterworks Equipment." For practical advice on selecting the correct material for your equipment, consult our Technical Department. Their services are freely available. *The American Brass Company, Waterbury 20, Connecticut. In Canada: Anaconda American Brass Ltd., New Toronto, Ont.*

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CORROSION-RESISTANT

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"...has stood the test of time..."

At Houston's San Jacinto Water System, W & T equipment — some of which is shown above — "has stood the test of time and has been used under many adverse conditions and always with satisfactory performance," according to Mr. Clyde R. Harvill, Superintendent. This dependability and long life result not only from proper design, but from the selection of the right parts and materials for every specific purpose.

Since 1913, W & T research has consistently sought out new designs, principles and materials. Every new development is placed under rigid field tests to prove it can meet the exacting standards that dependable and economical chlorination demands.

When you depend on W & T equipment, you have the assurance that 40 years of experience in the chlorination field is being used to bring you the best in design, parts and materials — selected to stand the test of time.

"Make Your First Choice Equipment That Lasts."

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Frank C. Amsbary Jr., President 1955–1956

Journal
AMERICAN WATER WORKS ASSOCIATION
VOL. 47 • JUNE 1955 • NO. 6

Municipal Management of Water Works in West Virginia

H. K. Gidley

A paper presented on Nov. 9, 1954, at the West Virginia Section Meeting, Huntington, W.Va., by H. K. Gidley, Director, Div. of San. Eng., State Dept. of Health, Charleston, W.Va.

THE author, after 24 years of experience with municipal water works management and plant operation in West Virginia, is still amazed at the diverse conditions observed in utilities. At one end are the many systems and plants well and efficiently managed by loyal, hard working, and underpaid employees, with little or no support or encouragement from the municipality. At the other extreme are the systems inexpertly, wastefully, and carelessly operated—again with the indifference of the communities served. Between these opposites are the many utilities where satisfactory conditions exist because of municipal interest, but where there is no assurance that the satisfactory state will continue. It is only fair to say that poor municipal management in West Virginia is found not only in water works, but is also common to other services, a situation which probably exists in other states, too.

Questionnaire

It was desired to know how municipal systems were managed in West Virginia, what water works people thought of the various management methods, and what were some of the common weaknesses of management. Answers to these and other questions were sought through a questionnaire mailed to selected persons connected with the 70 municipally owned water systems in the state. Some of the information gained through the questionnaire may not have a high degree of accuracy. The answers in many cases undoubtedly reflect personal bias. For example, a strong superintendent, nominally under the direction of the municipal governing body, who actually is free to run the utility without check or interference, would tend to support the existing situation, even though, in the long run, it may not be in the best interests of the city. Never-

theless, some interesting and valid conclusions clearly seem to have been obtained.

There must have been considerable interest in the questionnaire, for there was an unusual response. Out of 70 mailed, 63 were returned without any resort to a followup or a second appeal. Those who answered the questionnaires were not supposed to sign them, but many did so. One person answered by phone, stating that he did not mind conveying the information in person, but that he did not think it wise to express himself on paper. The survey results are shown in Table 1, which gives the percentage of affirmative replies.

TABLE 1

Management Survey Results

	per cent
1. Prime responsibility for municipal water works management (63 reporting)	
a. City council (or committee of council)	73.0
b. Appointed board (separate from council)	17.4
c. Elected board (separate from council)	6.4
d. Others	3.2
2. Water system operations directed by (63 reporting)	
a. Superintendent having overall charge	61.9
b. Separate department heads reporting to council or board	8.0
c. Mayor, council committee chairman, or city manager	30.1
3. Faults of municipal management of water works (41 out of 63 reported 1 or more faults)	
a. Water works income used to support other municipal functions	19.0
b. Lack of interest (mayor and council)	15.9
c. Inexperience of mayor and council in supply matters	41.3
d. Too frequent change of mayor and council	31.7
4. Municipal water works plan favored (59 reporting)	
a. Superintendent responsible to mayor (or council)	50.8
b. Superintendent responsible to appointed water board (separate from council	
c. Superintendent responsible to elected board (separate from council)	15.3

Responsibility and Direction

Most West Virginia cities and towns are governed by strong mayor-council administrations, and only a few employ city managers as the chief executive officer. The survey results show that in 73 per cent of the cases the prime responsibility for looking after the municipal water work is vested in the elected governing body of the city. It is common practice for the mayor to form within the council a water committee which handles much of the detail work. In a few cities, the systems are managed by special boards which are either appointed (17.4 per cent) or elected (6.4 per cent). The members are usually appointed or elected for relatively long, staggered terms of office. In some instances, the mayor may be an ex officio member of the board. One arrangement which is listed under the "other" category may be worth mentioning because of its oddity. The city involved has a char-

ter which provides for government by a board of four directors, two from each of the major political parties. After election, for concurrent 4 year terms, the board holds an executive session and selects from its own body a mayor, a water director, a director of streets, and a treasurer.

As expected, most municipal systems are under the direction of an overall superintendent (61.9 per cent). It is also significant that almost one-third (30.1 per cent) of the systems (usually small) are administered directly by the mayor or council. A good deal of the mismanagement, waste, and ill-conceived planning in municipal water works affairs results from either [1] well intentioned but uninformed elected officials who endeavor to take over the active direction of operations; or [2] officials who are seemingly indifferent to conditions and permit some incompetent employee to neglect and misdirect the operations. A few of the larger cities operate water systems without an overall manager, but employ several separate department heads, each of whom reports directly to the water board or council.

Municipal Faults

It is of interest to note that 65 per cent were willing to classify the municipal management of their systems as faulty in one or more respects. Five of the 22 who listed no faults marked "none" in the appropriate section of the form, but seventeen did not qualify their silence. Possibly, some of the latter did not wish to be critical of their own management.

The author is not qualified as a tax specialist, and so will not comment on the principle of levying a general-purpose tax on water sales, for that is precisely what occurs when water funds are diverted for nonwater supply usage. The "robbing" of water funds cannot be condemned too strongly when such diversion results in an inadequate, deteriorated system. This situation exists in several communities in West Virginia. Mayors and councils may be legally responsible for water works management, but some of these governing bodies have little or no interest in the systems, and the disinterest contributes to and invites deterioration. According to the survey, almost one city administration out of six is reported as having little interest in the local water system.

Inexperience seems to be the most common indictment of elected municipal officials who look after the supply. There is such a large and frequent turnover of elected officials in some cities that the superintendent is continually handicapped in his dealings with the governing body. This situation largely accounts for the short-sighted, timid approach many cities take in supply programs. Communities which have flourished for a hundred years, and will obviously be in existence a hundred years hence, are too often saddled with 10-year administrative thinking in water supply. Hesitancy in making needed capital improvements, except under emergency conditions, is not at all uncommon in municipal water works management. Moreover, there is often the uneconomic extension of water lines for political or personal reasons. The finances of small systems are often strained by these nonproductive investments.

Although 73 per cent of the public systems are administered by the mayor-council authority, the survey shows that only 50 per cent favor this form. Many of the questionnaires

were undoubtedly answered by individuals who would be prejudiced in favor of the prevailing system, regardless of merit, but the results indicate considerable dissatisfaction with mayor-council administration. About one-half of those who indicated a choice of administration favored a water board separate from the mayor-council, with 33.9 per cent desiring an appointed board, and 15.3 per cent preferring an elected board. The author favors a separate board, with a superintendent or general manager responsible to the group.

Summary and Conclusions

A survey of municipal water works management in West Virginia revealed that about three-fourths of the systems are administered by the mayor and council, and approximately one-fourth are under the guidance of an elected or appointed water board. Although two-thirds of the systems are headed by a superintendent, the remaining one-third are managed directly by the mayor.

One of the faults of municipal management is the diversion of water works income for other municipal uses, to the extent that the water system is adversely affected. Other serious and common complaints are that lack of interest, inexperience, and frequent turnover of councils badly cripples efficient management. There is considerable objection to the mayor-council form of water works administration, about one-third of those queried who operate under that system favoring an administration under a separate water board.

The survey results support the author's observations that many municipally owned systems in West Virginia suffer badly because of mismanagement, neglect, and abuse. Municipal organizations, state agencies, and the water works industry should initiate a study of the management problem in West Virginia, and should seek to establish a legal framework of management and operation standards which would prevent gross neglect and abuse of publically owned systems.

Financing Extensions in Maryland and Delaware

Panel Discussion

A panel discussion presented on Oct. 28, 1954, at the Chesapeake Section Meeting, Baltimore, Md.

Charles B. Wheeler

Chief Engr., Baltimore County Metropolitan Dist., Towson, Md.

THE financing of water works extensions in Baltimore County, Md., is carried out by the Metropolitan District. It was created by an act of the legislature in 1924 to provide for establishment of a water supply and construction, maintenance, and operation of sanitary sewage and storm water systems. Long-term bonds were issued to obtain funds for these purposes; taxes, assessments, and benefit charges were levied for the payment of the securities. The act established a self-supporting financial structure separate from that of the county which is maintained from general tax fund receipts. The 1924 act states, in part:

> At no time shall the total issue of bonds outstanding for any purpose, under this Act, exceed 7 per cent of the total assessable basis of real and tangible personal property assessed for taxation purposes, within the Metropolitan District. The payment of interest on, and the retirement of, said bonds shall be provided out of funds received under the provisions of . . . Section 335,* and, to the extent necessary, by a levy annually against all taxable property in the Metropolitan District less the connection charges, other available funds, front-foot assessments on hand and/or estimated to be received during such year applicable to the payment of interest and retirement expenses, for all bonds issued, provided that in the event such sum so levied shall not provide a sufficient sinking fund to retire all such bonds as they mature, then the Commissioners shall levy annually against all taxable property in Baltimore County a sufficient sum to make up the deficiency.

* This section provides for the collection of water supply and sewerage connection charges, annual front-foot benefit assessment charges, and annual sewer service charges.

Financial Requirements

The act specifies that every installation contemplated under it must be shown to be financially self-supporting within a reasonable period. Otherwise, the commissioners are not permitted to approve the project unless the property owners requesting it pay the unsupported amount.

The district must finance both revenue-producing and nonrevenue-producing extensions. The former consist principally of local main extensions, usually from a nearby source of supply. Nonrevenue projects involve distribution system improvements, such as elevated water storage tanks, feeder mains, pumping stations, and the reinforcement of mains between two separate systems.

Revenue-producing work is financed from front-foot assessment charges.

Where there is insufficient income, nonreturnable contributions are required by law from the interested property owners. Front-foot assessments are applied, according to five property classifications, at the rate in effect at the time the extension is made. Originally the charge, collected on an annual basis, was established at 15 cents per foot. As construction costs increased, it became necessary to raise the front-foot rate. The current subdivision assessment is 25 cents per foot. Industrial and business establishments are charged an additional 9 cents per foot. Agricultural property is assessed at the 25-cent rate, but only for 150 ft when the owner makes the connections. For small acreage not used for agriculture, a sliding scale is in effect. The total charge depends on the length of main abutting the property frontage. The classification of "remote property," for installations which cannot be financed at the normal rate, carries an assessment of 30 cents per foot. A definition of this classification follows: "Property which the chief engineer of the Metropolitan District shall determine, subject to approval of the county commissioners, to be so far removed or situated in respect to existing water facilities that the cost of the installation is prohibitive under the normal rate, shall be classified as remote property."

Extinguishment of Charges

Front-foot charges appear on the individual property tax bill and are collected annually in advance through the treasurer's office. These bear the same interest and are subject to the same penalties in the event of nonpayment as do the county property taxes. The length of time the assessments are to be in effect for any particular installation has not been established by law. It is expected that the charges will be collected from a property owner until the retirement of the bonds issued in the year of construction. The annual assessment may be extinguished if desired by the owner. This must be done within 1 year from the first levy. The amount to be paid is based on the capitalized assessment charge plus compensation for the loss of interest. The present cost of extinguishment is $5.50 per foot. Thus, an owner of 100 ft of assessable frontage could pay $550 to be relieved of the annual charge.

Front-foot assessments are capitalized at 5 per cent in determining if a proposed installation is self-supporting. For example, 100 ft of frontage at the 25-cent rate would support $500 of construction cost or produce a financed income of $5.00 per foot. Therefore, if full assessment is available on both sides of the street, a construction cost of $10.00 per foot can be sustained. Usually, this is not available, however, because of intersecting thoroughfares and corner or irregular lots. Moreover, a street may be developed or subdivided on one side, the opposite side producing little or no immediate income. If there is not sufficient developed property to support the cost of construction, the interested owners desiring the extension must make up the deficit.

Refunds

An arrangement has recently been devised to permit recovery of the deficit contribution on installations not presently self-supporting where additional properties may become assessable. Briefly, the deficit extension agreement provides for refunding all or a portion of the contribution according to the financed amount of additional assessment that becomes available during the life of the compact.

Previously, it was necessary for an individual to advance the total cost of the installation in order to gain recovery of the unsupported amount. This deposit was made under terms of a service extension agreement which called for the return of the financed income from assessments as service was taken and the annual charges were received. Formerly, all builders and developers requesting utility installations were required to enter into such an agreement. This was done principally to insure the district against financial loss in the event a new project did not prosper. Installations are now made through a public-works agreement, with the developer contributing any unsupported deficit.

Nonrevenue work, mostly distribution system improvements, must also be financed by the district. Although the law makes no distinction for types of usage and does not provide for separation of funds applied to revenue or nonrevenue operations, it seems obvious that the latter must be paid for from sources other than front-foot assessments. Apparently, the original drafters of the District Act did not foresee the tremendous volume of nonrevenue activities necessary to maintain adequate water and sewer service for the rapidly expanding community areas in Baltimore County, one of the fastest growing areas in the United States. Arrangement was made, however, to levy an ad valorem tax against all property in the district to the extent necessary for the financing and retirement at maturity of outstanding bonds when there would otherwise be insufficient means. This charge varied from 10 cents per 100 dollars of property valuation in 1938 to 3 cents in 1941–49. If the ad valorem impost does not produce a sufficient amount, the commissioners are empowered to raise from all taxable property in the county a sum large enough to make up the deficiency. The ad valorem tax has not been in effect since 1949, so that the cost of nonrevenue work is now met mainly from reserves and sewer service charge account profits.

Another source of money for nonrevenue activities is service charges applied to the individual water consumption bill. The city of Baltimore is required to maintain and operate the district distribution system and must bill and collect the water charges. In 1952 an agreement provided that a service charge was to be collected on all county water bills and that the profit, after deductions for city expenses chargeable to county operations, was to be made available to the district.

Conclusions

Some of the principal advantages of a properly regulated financing authority are as follows:

1. Water works extensions are on a self-supporting basis, tending to curb reckless installations requested by politically favored groups.

2. The individual property assessment system does not work to the disadvantage of those who receive no benefits. That is, a person who does not have service available does not have to pay a charge.

3. Construction bonds can be issued without the necessity of submitting the proposal to a general referendum.

Alfred Machis

Asst. to Chief Engr., Washington Suburban San. Com., Hyattsville, Md.

The Washington Suburban Sanitary Commission is a corporate body authorized by the legislature of Maryland to provide water, sewerage, drainage, and refuse disposal for the subur-

ban area of the state surrounding the District of Columbia. Created in 1918, the commission has been enlarged many times until at present it has jurisdiction over 212 sq miles. The rapid increase in population during the past few years has created many problems, not the least of which has been the raising of funds for a construction program.

In some respects, the financial structure of the commission is similar to that of the Baltimore County Metropolitan District described by Wheeler (see p. 523, this issue). In other respects there are important differences. The commission is autonomous and independent of the two bodies governing the counties in which it operates. The legislature granted it broad powers of imposing and collecting charges for water consumption, sewer usage, refuse collection and disposal, construction of house connections, and plumbing and gas inspection. In addition, a front-foot benefit charge and an ad valorem tax, collected by the counties on the tax bill, are remitted to the commission.

The basic elements of the financing plan were established in the original acts. Money for capital improvements is provided from bond issues, for which popular referendums are not required. Bonds may be issued, for certain purposes, up to 14 per cent of the assessed valuation of properties within the sanitary district. These purposes include: water and sewer extensions which are financed from general construction bonds without specific allocation; storm drains; incinerators; and major sewerage facilities. The general construction bonds may be issued without specific legislative authority. Bonds to provide funds for major water improvements beyond the scope of ordinary extensions are exempt from the 14 per cent limitation, but specific legislative authority is necessary.

Population Growth and Debt

The rapid population growth of the area has affected the bonded indebtedness of the commission. During a period when the physical size of the distribution system was about doubled, the outstanding indebtedness increased to 3½ times its former amount. In 1945 the overall debt was $22,000,000; by 1953 it had increased to over $74,000,000. At first glance, the amount appears to be excessive, but it must be realized that inflation was a contributory factor. A more realistic comparison is secured when the increase in the assessed value of properties within the district is taken into account. In 1945 the outstanding obligations represented 10 per cent of the assessed value; by the end of 1953 they had decreased to 9 per cent. It appears that the growth has been healthy.

Since its creation, the commission has been faced with a problem not usually found in most municipal water supplies. New developments have been almost entirely residential, with virtually no industrial and only a moderate amount of commercial growth in recent years. The supply and distribution systems of the commission have been independent of the District of Columbia, the nucleus of the population growth. Its dispersed character, together with the lack of a wealthy core to support the initial investment in reservoirs and treatment facilities, made it mandatory to evolve an appropriate financial plan.

Its basic philosophy was the division of the operations into a number of units, each of which could be made self-supporting. The recipient of benefits paid only the charges for services received. Both the water and sewer

extensions are financed from general construction bonds. These are redeemed from front-foot charges upon the adjacent property. The rates fixed each year are dependent upon the total cost of the extensions and the amount of assessable frontage. Consequently, the new extensions are self-supporting, so that old customers are not required to pay for them.

To keep the annual debt service cost at a point where the area could bear the burden, the first securities issued by the commission were term bonds of 50 years. Some time later, the legislature required that all bonds be of the serial type with a maximum maturity in 40 years. By then, the commission had grown sufficiently to finance its construction in this manner. Since 1952 the commission has further reduced the maximum life of new issues of general construction bonds to 30 years, resulting in the saving of interest payments.

From time to time, requests are made for extensions which engineering study indicates would not be self-supporting. Where the excess cost is not great and the extension obviously is desirable, the commission may authorize the project on the basis that the deficit will be balanced by a surplus from other projects. Sometimes development is contemplated in an area beyond the perimeter of the present system. If this is desirable but would not be self-supporting because of prematurity, the applicant may be required to pay the deficit for the premature period. A stipulated cash contribution would be required before the authorization of construction.

Merits of Public Debentures

Financing extensions by public-obligation bonds has certain advantages, together with certain disadvantages. It encourages the development of new areas by lowering necessary capital outlay of the builder. The self-sustaining requirement for each individual project induces the growth to proceed in an orderly fashion. There is also the benefit inherent in an integrated system of adequate supply, treatment, and storage facilities. As a consequence of the greater security obtained in spreading the obligation over a large number of residents, the interest charges are generally appreciably lower for a public utility than each resident could command alone if the cost of the water and sewer installations were in the purchase price of the house and were financed by a mortgage. A saving to the individual results. On the other hand, the public is shouldering a part of the risk. This is not too apparent so long as the area continues to grow vigorously and the economy is healthy.

The alternative to public financing is the requirement that each development pay the cost of extensions. Thus, a homeowner would ultimately pay the cost of the extensions in the price of the property. As the builder bears the risk of selling his construction in a competitive market, it should be necessary that he choose the location with great care. Being relieved of the risk as well as the increased indebtedness, the public would conceivably be in a better position to finance other necessary ventures. Adequate supply and fire protection may require some mains of greater than normal size. The developer and immediately adjoining property owner can hardly be expected to pay for this, so some relief ought to be considered. One way might be to make the price of the mains in excess of the cost of an equivalent 8-in. main a general public obligation, the balance to be paid by the builder. The

commission has considered this a possible means of reducing borrowing requirements, but as yet has made no change in the original plan.

Judging Feasibility

To clarify the mode of determining if an extension is economically feasible, it may be desirable to review the procedure through which an application passes. A preliminary layout of the lines is made. From this plan an accurate estimate of quantities and cost may be gathered. On the same plan the proposed area is plotted and the frontage abutting the mains is determined. The annual yield at the current rate is compared with the yearly debt service of the applicable bonds. If the project is self-sustaining, it may be authorized for construction; if it is not, then preconstruction contributions may be required. At the close of each year and prior to the first billing of new front-foot benefit charges, the commission establishes a new rate sufficient to fund the expense of all extensions built during the year. From year to year the front-foot rate does not vary greatly. In fact, it has been held at the same basic subdivision figure of 25 cents per assessable foot since 1952 and has increased only 5 cents in the last 6 years.

The commission is obliged by law to divide all property abutting its mains into four groups: agricultural, small acreage, industrial or business, and subdivision. The front-foot charges are fixed in accordance with the classification. Agricultural property is exempt until a service connection is provided, at which time the charges are limited to a maximum of 300 ft. The present schedule ranges from 33 cents per foot for the first 300 ft in the business group to 7 cents per foot for all in excess of 300 ft in the small-acreage group.

By financing water and sewer extensions with general construction bonds, refunded from front-foot benefit charges, the commission has removed the burden from the present consumers. The water rate reflects only the expense of supplying, treating, pumping, maintaining, and operating the system, together with the annual debt service on the major improvements, such as the treatment plants, pumping and storage facilities, and feeder mains. This is more equitable than recovering all costs from the water rate as the consumer is not necessarily the same individual who benefits from the construction of extensions.

Security of Bonds

The bonds are secured by a legislative act which requires the commission to fix annually the water rate to produce sufficient revenue to meet the requirements of outstanding bonds. This increases the surety and reduces the interest rate as the securities issued for the construction of water and sewer extensions are repaid from front-foot benefits. An additional guarantee is that each county must pledge to use its power of ad valorem taxation to further improve the firmness of the bonds and reduce the interest.

Imposing the full load of payment for extensions upon the front-foot charge would fall somewhat short of placing the cost upon the recipient of the benefits. First of all, improved property would gain somewhat more than an adjacent unimproved lot. Further, the nearby real estate would derive a gain in living conditions and a more healthful environment, even though it did not as yet abut a water main or sewer line. The construction would bring closer the time when further extension could serve nearby

areas. These intangibles, though difficult to assess exactly, were recognized in the initial financing. The plan authorized by the legislature empowered the commission to apply the receipts of an ad valorem tax toward the requirements of its bond program.

There are certain nonassessable footages of water main which must be financed in some equitable fashion. These include mains crossing parks, street intersections, and various nonconnected agricultural lands. The lines are frequently an essential part of the overall system necessary to serve future extensions and so could equitably be paid from an ad valorem tax. Some of the major works, such as treatment facilities, pumping stations, and feeder mains, are an asset to the entire community, which should bear the expense. These costs could also be appropriately met from the ad valorem tax. This levy has been set at a modest 10 cents per $100 of assessed valuation. In 1953, the yield tax was over $900,000. Six cents of the rate is applicable to water and sewer system costs, the balance is applicable to the construction and maintenance of the storm drainage system. The proportional part of the ad valorem tax yield was about one-fourth that obtained from the front-foot benefit charges. The annual cost of water and sewer service to a typical resident is shown in Table 1.

TABLE 1

Typical Annual Cost of Water and Sewer Service

Item	Amount
Water consumed, 60,000 gal at 27¢/1,000 gal	$16.20
Meter service charge, ¾-in. meter	2.00
Front-foot benefit for water main, 60 ft at 25¢/ft	15.00
Sewer usage charge, 60,000 gal at 14¢/1,000 gal	8.40
Front-foot benefit for sewer line, 60 ft at 25¢/ft	15.00
Ad valorem tax, $8,000 at 10¢/$100	8.00
Total	$64.60

Water Supply Bonds

In addition to the extension of the lateral system, the commission has had to provide the requisite supply, treatment, pumping, and major feeder mains. The funds needed for these items also have been obtained from bond issues, but from the kind termed water supply bonds. The magnitude of the construction involved may be gleaned from a comparison of the change in the bonded indebtedness from $1,250,000 in 1940 to $17,000,000 in 1953. The revenues for the redemption and interest requirements of these securities are entirely different from those of the general construction bonds. It would be grossly unfair to lay the system-wide expenses to the new consumer, for all benefit from the improvements. The most equitable measure of benefit derived is the consumption. Thus, water revenues are applied to the retirement of the debt. In 1954 about 10 cents of the total charge of 27 cents per 1,000 gal was necessary for the retirement of the bonds.

W. Compton Wills

Mgr. & Chief Engr., Board of Water Comrs., Wilmington, Del.

Financing water works extensions within the corporate limits of Wilmington, Del., poses no unusual problem for the Board of Water Commissioners. The area is fairly regular in shape, and the early water engineers wisely provided for a good many large supply mains of 16–48 in. The scheme of distribution has continued a long-range plan of feeder extensions and the development of an 8-in. grid sys-

tem, the size adopted 30 years ago as the minimum installation.

State law insists on rates adequate to pay the interest on the water bonds and the running expenses of the water works. Funds are readily available for extensions in the city. It installs at its own expense the mains and services to the curb lines. Since 1914 the services have been 100 per cent metered, the cost being borne by the city. On occasion, extensions have been longer than the income warranted so that an annual interest charge has had to be made until the receipts produced sufficient return on the investment.

Beyond the metropolitan boundaries it has been more difficult to plan for and extend service. One school of thought desired to go after the customers as the population started moving to the suburban area. An opposite, widespread feeling was the preference to stay within the city limits because the spread of facilities would retard and discourage expansions of the boundaries. A majority gave considerable support and encouragement to the extension of service to the rural areas. On occasion, the private suburban companies felt that their section was being invaded, but on the other hand, there was good reason to believe the reverse was true. The tremendous rural growth experienced recently no longer permits a utility to think of, voice a claim to, or harbor anxiety over any certain territory. In fact, there is silent approbation upon learning of a utility bold enough to absorb a development or new area in which the city water department at one time had an interest.

It has taken twenty years for Wilmington to climb from the introductory or exploratory level of rural water supply to a state of rapid expansion. The start was made by the acquisition of a few hundred feet of existing 8-in. main and has grown to the ownership and operation of many miles of lines of various sizes.

Ten-Mile Distribution Limit

The state allows the Wilmington Board of Water Commissioners—in addition to its power and authority in the corporate limits—to distribute water through all of the territory within 10 miles of the city's boundary. The previous limit had been 1 mile. The right was also given the board to determine the persons, firms, or corporations to which it should dispense water. Further, permission to fix rates was granted, as well as freedom to regulate the use of the water and to supply it upon terms and conditions agreed upon by the board and the consumers.

Outside of the city, extensions and service pipe installations are made at the expense of the developer. Within the last 4 years, meters have been installed at the consumer's expense. Ownership of the mains, services, and meters has been retained by the city, so that maintenance, repairs, and operation of these facilities have been at its expense. During the past 18 years, Wilmington has obtained a donated capital investment of $1,342,000. This acquisition of facilities by donation may seem rather severe, but it always is the individual homeowner who really settles the main installation bill. If he does not pay it to the city, directly or otherwise, he must pay it to the developer, who retains title to the water system. One of these, recently, had to be bought by the municipality in order to reach and serve other territory. Thus, the developer was paid twice for the system: once by the individual

homeowners through lot assessment at time of purchase, and again by the city's acquisition. The builders, not desiring to enter the water supply field, have agreed to present the lines to the municipality.

Rates and Main Sizes

Except to manufacturers, a double rate is applicable in outside areas. As the average ⅝-in. meter annual minimum rate within the city is $14, less 5 per cent discount for payment within a specified time, the charge at the double figure has proved to be no higher than that of private suppliers in the territory. Thus far, the rural business has been found profitable. Moreover, the metropolitan consumer or taxpayer benefits from the larger water works made possible by the combined revenues from the greater territory. The increased facilities in Wilmington are reflected in additional fire protection.

The WPA and PWA gave a good deal of assistance to the first sizeable extensions to the outlying sections. The federal administrations were active at a time when the area badly needed and vigorously sought the municipal service. Development of the country areas continued thereafter at a fairly rapid pace. During the early days, the old-fashioned idea of 2-in. main installations with about twenty houses to each 2-in. extension was followed. At first, the rural dweller asked for just enough water to replace a well. Then it became evident that fire protection brought about savings and advantages. The city soon learned that only the best installation would in the end be considered satisfactory and that the minimum pipe size should be 6-in. A rule embodying this principle was made, soon to be followed by a general demand for 8-in. mains, making the sizes comparable with metropolitan installations.

The rapid growth has caught up to some of the earlier planning. Swift and sudden expansion of areas and mushrooming of developments by residential, industrial, and commercial building, coupled with plans for highway revisions, combine to set up constant problems. These are reflected to a marked degree in the financing. Highway plans are important because they bring up many costly problems involving the distribution system. Roads are now being extensively rebuilt. It has been Wilmington's good fortune for many years to receive reimbursement for alterations of existing pipelines and appurtenances. Unfortunately, the department must spend the time to make the changes. Further, a large investment is placed in the ground for improvements, not to be used advantageously for a long time to come. Efforts are now being made to locate, as nearly as possible, new installations in conformity with future plans for highway changes.

Expanding Carefully

Rural expansions of a municipal supply can get out of hand, as in Wilmington, where the city's water is much in demand. The extensions should be made slowly and only in conformity with a comprehensive, long-range plan. The area should be well explored to learn to what future extent the demand will increase. The first extensions to the outlying districts do not represent excessive investment—they are readily served by the existing plant. As the territory served grows larger and big users appear, new facilities, often quite expensive, become necessary. Of course, such matters can be provided for in the rate structure.

In one rural section where a short time ago a few hundred persons were served by the department, there are today about 7,000 consumers. The area will probably increase to 40,000 people, with an additional 80,000 people in the adjoining sections.

Experience has shown that rural-supply plans can be upset by an unexpected moving in of large industrial consumers. Such a problem has occurred more than once in Wilmington. As opposed to the orderly growth allowed by housing developments, new industries make demands on a system which necessitate a complete revision of the plant. Constant inspection is needed to make certain that a planned, systematic expansion does not get out of hand, particularly when extending to open areas. When unexpected changes occur, the board slows down its increases until the comprehensive scheme can be set to allow for service to the unforeseen, large-scale user newly entering the region.

Summary and Conclusions

Many of the ideas discussed previously may seem to be the everyday thoughts of the private rural supplier accustomed to open-area planning. As they also apply, however, to a municipal supply, they are important and special considerations. The metropolitan system has its primary interest within the town boundaries. The secondary point of service for adjacent territory is very important in the overall picture of the city's welfare. Extensions should not compete with established utilities and should only be located in areas likely soon to become a part of the city, with assured adequate financial return. Perhaps the difficulties presented are the usual ones which arise in the middle ground between providing a small amount of rural service and meeting the needs of an extended metropolitan district. Again, the problems brought forth may be those which usually appear when boundaries cease to expand and services are in great demand beyond the limits.

There is no perplexity in the financing of extensions in Wilmington where the territory is so fully developed that little or no construction is possible. Beyond the metropolitan borders, growth is enormous and, despite donated costs of distribution installations, plant investment is becoming larger. A close watch is being kept on rate schedules to assure that rural service is beneficial to Wilmington. No complete policy has yet been evolved for the city.

Coping With Summer Distribution Problems in Dallas

J. D. Henry

A paper presented on Oct. 19, 1954, at the Southwest Section Meeting, El Paso, Tex., by J. D. Henry, Supervisor of Distr., Dallas Water Dept., Dallas, Tex.

DALLAS has experienced a phenomenal growth in the past 10 years, and the distribution system has keenly felt the increase. For example, as part of the planned construction program a 30-in. main was built to serve a certain area. The population increase was so rapid, however, that an additional 20-in. main was needed ahead of its planned installation time.

On one August day, when the temperature reached an unprecedented 111°F, unrestricted sprinkling, hundreds of air-conditioning units, and other sources of heavy water consumption caused pressure to drop below 10 psi at 7:30 PM on the highest hill in the area served by the 30-in. main.

The Dallas distribution system in 1954 had 1,773 miles of mains and 155,995 services and meters serving a population of approximately 550,000. Table 1 shows the growth of Dallas since 1944 and what the department has done to try to keep pace with the city's phenomenal development. The cost of expansion was financed mainly with bonds and prorated charges to the new customer.

The drop in consumption in 1952–53 was caused by rigid water use regulations brought on by the severe drought which depleted the supply in Lake Dallas, the primary reservoir. The 5-gpcd growth in 1954 over 1952 alone increased the annual consumption 912 mil gal, using the population figure for 1952. This rise has had a telling effect on a distribution system which was already strained because of the demand of an enlarged population.

Dallas Topography

The topography of Dallas makes it necessary to have three pumping zones:

1. East and west low-level service areas.
2. East and west high-level service areas.
3. North high-level service area.

The elevations range from 398 to 550 ft above sea level in the low-level service areas. The high-level service areas range from 500 to 698 ft above sea level. The elevations in themselves pose a pumping and distribution problem. For example, it is necessary to have 67 psi at el 398 in order to have 1 psi at el 550, without any drawdown whatsoever. Consequently, in the periods of peak demand there is 140 psi at some of the low points and just 10–15 psi at some of the high points. This is true only at the elevations where the size of the transmission mains is inadequate to maintain the

volume under a severe load. On some of the peak days, the demand has been 2¼ times that of a normal day. Also, the peak hour demand has been twice that of a peak day, taking the city as a whole. In some residential sections, such as the Oak Cliff area, the peak hour demand on some days has been three times that of the peak day in that section.

In the Oak Cliff high-level service area, the main booster pump station (Sunset station) is located at el 612 on the western side of the section it serves. Most of the eastern part, known as Trinity Heights and Lisbon, a predominantly residential area, is several miles from the Sunset station. The installation boosts out of a 15-mil gal ground storage reservoir which rides the west low-elevation system all the time.

The Trinity Heights and Lisbon area is supplemented by a 1-mil gal elevated storage tank, located at el 611, approximately the same height as the Sunset station. The tank is 35 ft tall, with a 71-ft riser, making the top 717 ft above sea level. With adequate transmission mains from the Sunset station to the tank, it would take only 46 psi at the station to keep the tank full of water at all times. There is another 1-mil gal elevated tank in the same district, but in the western part of the section near the Sunset station. This tank has a ground elevation of 688 ft and an overflow elevation of 783 ft. The hydraulic gradient of the higher tank is 66 ft above the lower. The difference in elevation necessitates an altitude valve on the lower tank to prevent its running over while the riser is not yet full at the higher.

TABLE 1

Population, Consumption, and Distribution System Increases in Dallas

Fiscal Year*	Population†	Mains Added *miles*	Mains in System *miles*	Services Added	Services in System	Water Use	
						mil gal	*gpcd*
1944–45	364,599	118	911	1,697	84,681	14,399	108
1945–46	378,572	50	961	4,370	89,051	16,085	116
1946–47	392,545	81	1,042	6,350	95,401	17,926	125
1947–48	406,518	75	1,117	8,262	103,663	19,589	132
1948–49	420,491	70	1,187	6,730	110,393	19,959	130
1949–50	434,462	93	1,280	9,632	120,025	19,013	120
1950–51	456,308‡	107	1,387	10,804	130,829	20,896	126
1951–52	500,000‡	161	1,547	8,251	139,080	24,575	135
1952–53	525,000‡	105	1,653	8,033	147,113	22,859	119
1953–54	550,000‡	120	1,773	8,882	155,995	28,103	140

* Oct. 1–Sep. 30.
† Federal census figures on each 10-year increase averaged among individual years.
‡ Based on chamber of commerce estimates and water department data.

Distribution Difficulties

Until the early part of 1954, a deep artesian well augmented the supply in the Trinity Heights–Lisbon section. Distribution trouble began in 1952–53 because of an increase in customers and demand. In late 1953 and early

1954 the Bonnieview booster station was built. The installation is located on the extreme eastern part of the section it serves. A 1,500-gpm pump takes suction out of a 16-in. transmission main in the low-elevation area and boosts the water into the high-elevation area. Even though the transmission mains were inadequate between the Sunset station and the Trinity Heights area, it was felt that, with the Bonnieview booster, the 1-mil gal elevated tank, the 1,500-gpm well, and a new 16-in. main built through Trinity Heights, the area could be adequately supplied in the summer of 1954. In the spring of 1954, however, the well supply failed. It was determined that it would cost about $37,000 to put the well back in good shape, and that it would take 3–4 months to finish the job. Because of the time element, it was decided not to restore the well immediately. In lieu of the well, a 1,400 gpm vertical submersible pump, housed in a buried steel drum with an inlet at the bottom and an outlet at the top, was installed to take water from a 12-in. main in the low area. In the early part of June 1954, a few days after the exceptionally hot weather began, the pump was put into service. At that time, the pump caused the pressure on the suction side to drop from 26 psi to 17 psi, but the 12-in. main was at the highest point (el 550) on the low area, and all the customers still had water. The main has a good volume because it is fed by a 24-in. and a 16-in. main a short distance away. As the days grew hotter and the weather became drier, however, trouble began developing.

With the heavy demand in June, the lower-elevation tank could not be kept full after 7 AM. On the first very hot day a loss of 15 ft from the tank could not be regained in time for the heavy load at 6 PM. At about 4 PM that day the tank valves were closed, taking it off the system. This dropped the pressure in the west high-level system from 39 to 29 psi. By 6 PM the pressure had dropped to 17 psi. At that hour the valves were opened slightly and water was doled out to the customers. There was 3 ft left in the tank when the heavy load went off, about 8:10 PM. During the 2-hr heavy-demand period, the pressure was kept above 18 psi on the highest hills in the immediate vicinity. Thus, all the sections in the whole west high area had at least 18 psi by continuing the operation until late September.

Remedial Measures

During the distribution difficulties, both the Bonnieview and submersible-pump boosters were being used. As the days became hotter, trouble began to develop on the high points in the low area near the boosters. Several recording gages were set on the elevated points, and it was determined that, by pinching the discharge valves on the two boosters, a certain amount of pressure could be maintained on the hills in the low area. In the meantime it became difficult to keep water in the new 2-mil gal elevated tank located in the area northeast of White Rock Lake. The following procedure was set up to overcome the trouble in all the sections in this manner:

1. A man was scheduled to take the tank at el 611 off the system before 7 AM, to check pressures intermittently all day, to return at 5:30 PM and to start doling the water out, to put the tank fully on the system about 8:10–8:30 PM, and then to stop work.

2. Another man was scheduled to go to the Bonnieview booster at 5:45 PM, and to pinch the discharge valve so that suction pressure remained at 32 psi. As soon as the pressure built up to more than 32 psi, he would open the valve and quit work.

3. A man was stationed at the submersible pump to keep the suction pressure above 14 psi, in the same way as described for Step 2.

4. The White Rock tank was to be operated similarly to the tank in Step 1.

On Jul. 24, 1954, there was a maximum consumption of 160 mil gal. On the 2 days following, more than 150 mgd was consumed. This so depleted the reservoir storage that alternate-day sprinkling regulations were put into effect on Jul. 27, 1954. These regulations reduced consumption about 7–9 mgd. The reduction was just barely enough to keep the system out of trouble on the extremely hot days that followed. The restrictions were lifted in the latter part of September 1954.

Improvements Under Way

Of the 1,773 miles of mains in the Dallas system, 307 miles range in size from 12 in. to 66 in. in diameter. More than 17 per cent of the system consists of transmission or feeder mains. A 48-in. transmission main is being built to supply water directly into the 15-mil gal reservoir at the Sunset station, supplement the west low-level area, and furnish more water for the west high-level area. Also, contracts are being let for a 24-in. main to supplement the White Rock area, and a 42-in. and a 16-in. main to supplement the Trinity Heights area. These improvements are expected to ease the situation in the critical areas by summer 1955.

Measuring Yield of Drilled Wells Without Pumping

Abraham Katz

A contribution to the Journal by Abraham Katz, Cons. Engr., Tel Aviv, Israel.

THE object of the method presented here is the measurement of the yield of a drilled well without actually pumping water. A test arrangement simulating pumping, or in some cases pumping during well development, provides information on well capacities when the degree of change in the water level is noted.

Wells fall into three general categories, from the point of view of the water-bearing strata to be exploited: [1] aquiferous sands, for which screen or gravel filters, or both, are used; [2] sandstone, gravel beds, fossils, and similar strata, for which perforated casings are often used; and [3] stable formations of limestone or chalk of Turonian or Cenomanian age (both Upper Cretaceous), especially the latter.

Filter installations require little well development. Yield is highest at the beginning, decreases somewhat during the first months of pumping, and remains stable for some years, decreasing toward the end, as the screen openings become clogged (if the screen does not tear first). When pumping from gravel or sandstone wells, the highest yield is obtained after development of the well—that is, after loose sand is removed from the stone cracks and fissures around the well to permit free percolation of water into the well. In Turonian-Cenomanian strata, well development rarely takes place unless the formations are relatively soft. Hard formations of this type practically do not increase their yield with pumping, as the water flows through wide cracks, freely parting material, and cavities.

The new method is intended particularly to measure the yield of wells of the third group, which do not require development. The method also has another application. In wells of the second group, which require development, it helps determine the permeability of the aquiferous strata. Whereas the latter benefit is more of theoretical interest, the determination of actual well yield has immediate practical value. Measurements can be carried out while drilling operations are still in progress. The results may determine whether drilling is to be stopped or continued. As the yield of wells equipped with filters changes little with development, the same method could be used to determine all the necessary data for the permanent pump to be installed in the well.

Primitive Test Method

Every well driller carries out a primitive pumping test when he believes that he has reached water-bearing strata.

He sees whether the water level in the well does not drop at all, or only to a very small extent, when he bails out water and the drill cuttings. By pulling out the bailer with water from the well several times consecutively, as fast as he can, he estimates the discharge. Naturally, the discharge determination is very inaccurate, owing to the slow speed of bailing.

Every time the bailer is lowered into the well, the driller attempts to check the water level to see if, and to what extent, it has receded. Where the water table is close to the surface, estimation is easier than where the table is fairly deep. The time required to hoist the bailer is rather long, and by the time the bailer can be lowered, the water in the well may have returned to its original static level, even if the water-bearing formations are not very permeable. This makes it difficult to estimate the yield, even roughly.

To overcome this difficulty, the drillers lower the bailer into the water, fill it, hoist it fast to a point just above static water level, and immediately lower it again. By withdrawal of the volume of water contained in the bailer, the water level drops. If the splash of the bailer hitting the water on being lowered again can be heard, and the point on the rope is marked, a very rough approximation of the drawdown is obtained. This is a very crude method, too, as many important factors—especially time—are not measured accurately.

A forecast based on such crude procedure was usually considered sufficient (together with other general data and formation samples) to warrant installation of a test pump. Frequently the results of pumping tests decided upon by such forecasts did not justify the expense. Recently, in some very deep wells, considerable sums of money have been wasted through costly installations of test pumps which resulted in practically no yield. Pumps of 500–600-ft setting had to be installed, static water levels being at about that depth.

Attempts at New Method

By close observation, and through collaboration with well drillers in carrying out the previously mentioned crude experiments, the author formed the idea of a new method for measuring the yield of wells. One of the main difficulties of the previous measurements was in accurately determining the static water level, as well as the depth that it fell. This was solved by attaching an electric water level measuring device to a "bucket," consisting of a length of pipe, closed at the bottom, and equipped with a small foot valve. The apparatus is shown in Fig. 1.

The electric apparatus for measuring water level is composed of a dry-cell battery (one pole connected to the well casing, the other to a voltmeter) and a single-wire insulated conductor with a free end that is lowered with the bucket. The electric wire is led inside the bucket to its bottom (to prevent damage to the wire while the bucket is lowered) and then through an insulated plug in the bucket bottom. The open end of the wire extends just below the plug. The electrical circuit is closed only when the wire end contacts water, the voltmeter indicating when this happens.

The electric device is first used to ascertain the static water level. Also, the exact point of circuit interruption and the corresponding water level are determined when the bucket is lifted. Reasoning along the line of the previously mentioned primitive method, it

was decided to lower the bucket into the water and then lift it at a high speed. When the bottom of the bucket came out of the water, the electric circuit would open, and the discharge could be determined at a certain drawdown. The water level decrease would be the difference between the static water level and the point of electrical interruption.

Difficulties of Early Method

Assume that the bucket is a 14-in. OD pipe 16½ ft long, containing about 133 gal, and that it is hoisted at a speed that would open the electric circuit after 2 sec. Further assume that the circuit opened after only 5 ft of the bucket was hoisted above the previously existing static water level. From this example, it might be thought that there was an actual pumpage (in gallons per minute) of $\frac{60}{2} \times 133$, equaling approximately 4,000 gpm, with a drawdown of $16\frac{1}{2} - 5 = 11\frac{1}{2}$ ft. This is not so. Even when using a pump, the drawdown is neither determinable nor stable during the first moments of withdrawal. In most wells, reasonable time is needed to obtain accurate figures for drawdown. Therefore, it cannot be expected that after a withdrawal period of 2 sec, the water level would be stable enough to permit accurate conclusions about the yield of the well. True, for an instant there was an indicated pumpage rate of about 4,000 gpm in the above example. Had the bucket been hoisted at a higher speed so that 1 sec elapsed before the open end of the wire broke contact with the water, it would supposedly have indicated a pumpage rate of 8,000 gpm. The drawdown would have been about the same, owing to the limited length of the bucket.

It is entirely doubtful, however, that the water-bearing strata of the well could yield such a discharge. There may be very little water in the aquifer. To make sure, the same volume would have to be taken out repeatedly and continuously with no lowering of the water level. This method would require a regular pumping test, which, of course, is what the author set out to eliminate.

Considerations in Devising Tests

After repeated measurements it became clear that the discharge measured by the new method is independent of the speed of pulling the bucket from the water. Pumped water is not being measured. The discharge rate measured here depends on the rate of replenishment of the well (the refilling of the "cavity" in the well that was created by drawing out the bucket with water). Pumping is an action that forces the water level to recede in comparison with the initial static water level. The rate of drawdown depends on the discharge and the quality of the aquiferous strata. The incoming water, which fills the "cavity" created by withdrawing the bucket, is not under any outside compulsion and depends only on the level potential and the quality of the strata. Therefore, measurement of replenishment is reliable, even if the time interval is short. During regular pumping from a well, water is drawn from a considerable area surrounding the well, forming a "funnel," and that space has to be replenished after pumping is stopped. In the author's test measurements, the well has to recover only a very limited volume of water, equivalent to the volume withdrawn by the bucket.

After regular pumping is stopped, the rate of water level rise is very fast

at first. A major cause is the return into the well of water contained in the pump column. The water rise decreases with approach to the initial static position. This is due to the fact that the water must fill more and more space in the funnel created by pumping, apart from the decreasing level potential. In other words, after regular pumping has been stopped, the slowing down of the rise in water level is not purely a function of level potentials, whereas in the new test measurements the only factor that diminishes the rate of water rise in the well is the decreasing level potential. Therefore, water level recovery in a test case is generally faster. Further, the rate of water level recovery after regular pumping is stopped provides only a nebulous idea of the flow of water from the aquiferous strata. The emptying into the well of the water contained in the pump column is a disturbing factor. In the author's method, the rate of water level recovery indicates exactly the volume of water that was returned from the aquifer to the well.

Further Test Procedures

There is no need to take into account actual pumping from the well, whether by regular bailing or by lifting the bucket with water just above the static water level and leaving it in the well. It is enough to measure the rate of recovery of the original static water level in the well, and draw all possible conclusions from it.

While measuring the rate of rise of water level, it is necessary to lift the bucket from the water faster than the rise of the water level. Care should be taken, however, not to lift the bottom of the bucket above the original static water level. The measured time period begins when the hoisting of the bucket starts and ends when the electric circuit is reestablished as the rising water level comes in contact with the bucket bottom. Then the length of bucket lifted is measured.

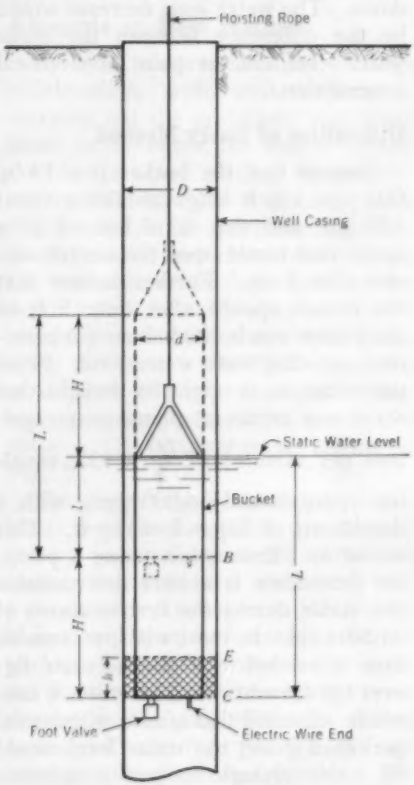

Fig. 1. Test Arrangement and Apparatus

The bucket is shown inside the well casing, with the position after hoisting indicated by broken lines. The symbols are explained on page 542.

It is known that if water from an outside source is poured back in a well in permeable strata, there will be a temporary water level rise in the well.

This is proportional to the volume of water poured back into the well, and inversely proportional to the permeability of the formation. Within a certain period after water has ceased being poured into the well, the water level, initially raised by the water added, will recede to its previous normal position. The less water added, the shorter will be this time interval. There are some exceptions, particularly in sandstone wells that require development. Such wells sometimes fill up and overflow when water is poured back into them, showing that the absorptive capacity of the aquiferous strata is almost nil. After development and cleaning out of the sand from the strata, such wells improve appreciably in yield.

Advanced Method

These considerations led the author to think that there was no need to pull the bucket of water out of the well. Similar results can be obtained by lowering into the well a bucket already filled with water. As the bucket enters the water in the well, there will be a rise in water level, which will recede quickly or slowly, depending on the permeability of the strata. The surplus water will flow into the formation spaces just as if water were added from the outside in a volume equal to that of the bucket, as measured by its outer diameter.

Considering the possibility that one can never forecast the quality of a well in advance, and that there are wells in which the formation samples seem to promise plenty of water, when in reality the water veins are clogged, it is necessary to carry out the first measurement with an empty bucket and draw out the water from the well. When the bucket is raised, a volume of water equal to that contained by the bucket will be drawn out. In a poor well, the rate of water level recovery, to fill the cavity created by the water withdrawn, will be slow. Were a full bucket to be lowered into such a well, the water level would rise above its initial static level and remain so, at least for some time, because the added water would not flow back readily into the formations. When the bucket is lifted, the water level will return to its initial static level, and this may create the misleading impression that the rate of rise was very fast and that the well yields much water. In subsequent measurements, which, of course, would only be carried out after the first test had shown that the well gave a good yield, the tests would be the same as if the bucket had been filled from the outside. The water level will recover after the bucket is hoisted.

Equipment Needed

Equipment needed for measurements includes:

1. An electric water level measuring device.

2. A "bucket" made of round pipe about 16–20 ft long, with an outside diameter as nearly as possible the same as the inside diameter of the well casing. The bucket should have a closed bottom fitted with a small foot valve (2 in. or 3 in.) and an insulated plug through which the wire of the electric measuring device extends. The longer the bucket, the more accurate will be the measurements.

3. A stop watch.

The insulated wire is led through the inside of the bucket, a bare end protruding through the insulated plug in the bottom. The bucket is lowered into the well, with the wire tied to the hoisting rope at intervals to prevent fouling. When the bare wire end

touches water (as shown by the voltmeter), the hoisting rope is marked at a corresponding point. The rope is slowly lowered further; water from the well enters the bucket through the foot valve; and, when the rope is lowered for a length equal to that of the bucket, it is marked again. The bucket is kept in the water in this position for several minutes, to stabilize the water level and to enable the water to return to its initial position, which may have changed owing to the insertion of the bucket.

Then the order is given to hoist the bucket to a preset point (not above the initial static water level). At the start of hoisting, the stop watch is set in motion, and the voltmeter is closely observed. If the electric circuit is opened (the bucket coming out of the water and the voltmeter showing zero), there is a wait until the water level rises and recloses the circuit. At that instant the watch is stopped.

Effectiveness

This measurement provides all the data needed to establish the rate of well replenishment by the aquifer. If it takes a long time for the electric circuit to close again, it is a sure sign that the well will have a poor yield. Drilling should then be continued, or the well should be abandoned if prospects of finding additional water-bearing strata are poor. If the time for reclosing the electric circuit is short, it means the well can yield much water, and drilling operations would not have to be continued.

When measuring yield by the new method, the actual size of the aquifer that feeds the well cannot be measured. Only conclusions about the quality of the water passages in the formations can be drawn. Therefore, where the existence of an underground basin of limited size is suspected, it would be advisable to carry out a prolonged pumping test. A short experiment of this type would not enable one to draw reliable conclusions. On the other hand, in districts well known hydrologically, the need for a pumping test is entirely eliminated by the use of the new method.

Calculations

Symbols used in the following calculations are: L, bucket length; d, outside diameter of bucket; $a = \frac{\pi d^2}{4}$, bucket cross-sectional area; D, inside diameter of well casing; $A = \frac{\pi D^2}{4}$, cross-sectional area inside the well casing; H, lifted height of the bucket; $l = L - H$, length of that part of the bucket that remained below the original water level; t, time in seconds to lift the bucket from Point C to Point B (Fig. 1); Q, well discharge; s, drawdown corresponding to discharge.

It should be noted that not all the water that refilled the space between points C and B was derived from the aquifer. The volume of water between the bucket and the well casing should be deducted. To facilitate calculation of the volume replenished by the aquifer, H is decreased by an amount h, the equivalent height, which, multiplied by the cross-sectional area of the inside of the well casing, would give a volume equal to that contained between the bucket and casing.

$$hA = (A - a)L \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$h = \frac{(A - a)L}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If there is no well casing in the vicinity of the static water level or if the casing extends only for a short length under the water level and is not pressed into the borehole tightly against the formations, the measurements are not accurate, and it would be difficult to draw correct conclusions. The value of A will be unknown if the borehole walls are not smooth. There may be large cracks, fissures, or cavities in the formations surrounding the borehole. As a result, h is indeterminate. As the size of A increases, h approaches L. With large cavities, h may almost be equal to L. It would then be practically impossible to measure the volume of water that would replenish the well after the bucket has been lifted (there would be no interruption in the electric circuit). Appearances, in such a case, would seem to indicate that there is a great deal of water in the well, whereas in reality there may be very little. To render measurements more exact in such applications, a bucket longer than usual must be used.

TABLE 1

Measurements at Kfar Saba Well *

Item	1st run	2nd run	3rd run
Measurements			
Lift of bucket (H)—*ft*	13.53	12.85	11.90
Time (t)—*sec*	5.00	4.50	3.95
Calculations			
Water level rise ($H-h$)—*ft*	11.08	10.38	9.40
Discharge (Q)—*gpm*	1,260	1,329	1,364
Bucket submersion after lifting (l)—*ft*	1.87	2.56	3.51
Drawdown (s†)—*ft*	7.40	7.75	8.24

* Data: L, 15.4 ft; d, 14 in.; D, 15¼ in.; a, 153 sq in.; A, 183 sq in.; $h = \frac{(A-a)L}{A}$, 2.48 ft.

† $s = l + \frac{H-h}{2}$.

Where there is a definite interruption in the electric circuit, positive conclusions may be drawn.

From Eq 2 it is learned that the size of a should always be as large as possible. That is, the diameter of the bucket should approach very nearly the inside diameter of the well casing. The volume of water replenished by the aquifer during the measurement is, therefore:

$$V = (H - h)A \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$Q = \frac{60V}{t}, \text{ the discharge per minute} \quad (4)$$

or:

$$Q = \frac{60\,(H-h)A}{t} \quad \ldots\ldots\ldots\ldots (5)$$

The units of volume or discharge would depend upon the units used for the dimensions. The discharge, Q, as calculated, corresponds to the average velocity of rise in water level between E and B. The volume between C and E was filled by the water originally contained between the bucket and the casing. Thus, the discharge, Q, is based on the average level potential between points E and B. Therefore:

$$s = l + \frac{H-h}{2} \quad \ldots\ldots\ldots (6)$$

The following examples involve measurements actually carried out by the author.

Kfar Saba Well

Table 1 indicates that, at a well east of Kfar Saba, the results of the measurements by the new method, as far as drawdown is concerned, do not agree with those obtained during the actual pumping test (shown in Table 2). At a discharge of 1,364 gpm, however, the results come closer. Therefore, it

seems that the drawdown in water level does not correspond to Q (well discharge). The rate of rise of water level at point E really should have been greater in the measurements at Kfar Saba (owing to the potential of the aquifer), but actually was not. The velocity of rise of water at the start has to overcome the inertia of the water. Therefore, the rate of rise at point E is slower than it would be if the water were already in motion before it reached point E. Thus, calculation of s becomes inaccurate, and, to a certain extent, so does that of Q (well discharge).

TABLE 2

Results of Actual Pumping Test at Kfar Saba Well

Item	1st run	2nd run	3rd run
Discharge—*gpm*	1,430	1,100	440
Drawdown—*ft*	5.58	3.38	0.66

Hittin Well

After drilling had reached a depth of 750 ft and aquiferous formations had been found, the author was asked by the owners to determine by the new method whether a pumping test was warranted at that point, or whether drilling should be continued deeper. The static water level in this well was at 510 ft. A pumping test at such a deep setting is very expensive.

After the measurements shown in Table 3 had been made, a pumping test was carried out. Because of the considerable depth of the water level, it was impossible to obtain, with the pump used, any discharge rates approaching those in the forecast (880 gpm). The highest pump discharge was 356 gpm, with a drawdown of 3.4 ft. Still, it was not clear why each of the author's three measurements yielded close to 880 gpm, but with different drawdowns. For estimating the discharge that could be pumped from this well, the measurements were satisfactory. In a report to the well owners, it was remarked that if the well had been in the Sharon Valley, an area well known hydrologically, a pump test would have been unnecessary.

TABLE 3

Measurements at Hittin Well *

Item	1st run	2nd run	3rd run
Measurements			
Lift of bucket (H)—*ft*	8.55	9.05	9.95
Time (t)—*sec*	4.0	4.3	4.9
Calculations			
Water level rise ($H-h$)—*ft*	6.13	6.62	7.55
Discharge (Q)—*gpm*	870	875	880
Bucket submersion after lifting (l)—*ft*	6.56	6.06	5.15
Drawdown (s†)—*ft*	9.65	9.36	8.92

* Data: L, 15.1 ft; d, 14 in.; D, 15½ in.; a, 153 sq in.; A, 183 sq in.; $h = \frac{(A-a)L}{A}$, 2.42 ft.

† $s = l + \frac{H-h}{2}$.

Sdeh Varbourg Well

Table 4 shows the results of two tests on a well in the Sharon Valley, where pumping tests have been dispensed with altogether. A permanent pump of about 2,000-gpm capacity has been ordered for this well, exclusively on the strength of the author's measurements. The first test was carried out after the well had reached a depth of 920 ft, when the drillers believed erroneously they had entered a rich water-bearing stratum. The second test was made after the well had reached 1,060 ft.

Comparing the two tests, it is seen that at a 1,060-ft depth, rich aquiferous strata were reached. The increase in yield was very considerable (about five times), although drawdown in both was about the same. Here again there are almost equal discharges (about 1,200 gpm), but for different drawdowns. There would be no great error, however, if, when selecting a pump for this well, a margin of a few additional feet in the length of the pump column were allowed. This would be of little consequence compared with the total cost of the equipment needed. The pump ordered for this well has been put in operation, and the results have fully justified the forecast.

TABLE 4

Measurements at Varbourg Well

Item	920-ft Depth			1,060-ft Depth	
	1st run	2nd run	3rd run	1st run	2nd run
Measurements					
Lift of bucket (H)—*ft*	12.00	15.44	17.10	15.93	14.13
Time (t)—*sec*	14.9	24.0	29.6	4.4	3.7
Calculations					
Water level rise ($H-h$)—*ft*	8.06	11.50	13.13	11.88	10.20
Discharge (Q)—*gpm*	236	209	193	1,188	1,197
Bucket submersion after lifting (l)—*ft*	10.30	6.85	5.21	6.36	8.16
Drawdown (s^*)—*ft*	14.33	12.63	11.80	12.35	13.28

* $s = l + \frac{H-h}{2}$.

Refinements of New Method

A further step toward a more accurate determination of drawdown may be taken. As mentioned before, the velocity of rise of the water decreases as the level rises, finally becoming zero when the level reaches the original static water level. Owing to this aspect only, different discharge rates should have been obtained, corresponding to the different drawdowns in the measurements. This expectation was not met in a long series of tests. In the Hittin well, a uniform discharge of 880 gpm was calculated. A similar occurrence was noted in the Sdeh Varbourg well (second test), when a uniform discharge rate of about 1,200 gpm was calculated. Even where varying discharges were noted, as in the well east of Kfar Saba or the first test at Sdeh Varbourg, they were out of all proportion to the differences in water level drawdowns that were measured.

It seems there are two factors interfering with accurate measurement, and if their influence could be eliminated, more accurate results could be obtained. One of the factors is the initial velocity of the rising water. The second is the slow starting speed at which the bucket is hoisted by the drilling machine. As already explained, the stop watch is set in motion as soon as the hoisting rope starts to rise. No doubt the speed of the rope is very much slower at the beginning, until the machine overcomes initial inertia and the hoisting speed be-

comes more or less uniform. The slow acceleration retards the rise of the water, yet the actuation of the stop watch has already started the time measurement.

Use of Two Electric Contacts

Elimination of these two factors is feasible if all measurements are carried out only when the water, as well as the bucket, is already in motion. This is made possible by providing two electric contacts below the bucket, at different heights, instead of just one. The measurements of the rise of water level will apply only to the interval between these two contacts. When lifting the bucket with the highest possible speed to a certain height, the stop watch is actuated only when water reaches the first contact point, and is stopped when water reaches the second contact point. Naturally, the two contacts must lie below the original static water level. In such a two-contact arrangement, the insulated conductor would consist of two wires, with two voltmeters, one for each contact point.

The distance between the two contacts and the sectional area inside the well casing are known, and the time is measured. These data permit accurate calculation of the aquifer replenishment, which corresponds to the potential of the average water level, midway between the two contacts. With this method, it is possible, by stopping the bucket (and the contacts) at different predetermined positions, to obtain an idea of the discharge rates at different drawdowns.

It should be noted that the time necessary for the water to rise between the two contacts could be very short. Therefore, measurement should be fast and accurate. It would be desirable to use a stop watch completing one revolution in 5–10 sec, compared with the usual 30–60 sec. For example, if the distance between the two contacts is fixed at 3 ft, and a discharge of about 1,200 gpm is expected, the time of measurement will be only about 1.1 sec. In such measurements, extra long buckets that displace the largest volumes of water possible should be used.

Modified Single-Contact Method

Results like those obtained from the two-contact method may be expected by the use of a modified single-contact procedure. Measurements are carried out as usual, but it is arranged to lift the bucket several times, with highest possible speed, to a number of predetermined but differing points. Fixed points are established at various depths below the original static water level. The time from start of hoisting till electrical contact is reestablished is measured in the usual way. The computations are made by taking into account differentials of the heights lifted, in relation to the corresponding time differentials. Calculations will then be as follows:

Difference in height of lifting the bucket:

$$\Delta H = H_1 - H_2 \ldots\ldots (7)$$

The time differential, in seconds:

$$\Delta t = t_1 - t_2 \ldots\ldots (8)$$

Volume of water that came up in that time interval:

$$\Delta V = \Delta HA \ldots\ldots (9)$$

The discharge per minute:

$$Q = \frac{60\Delta V}{\Delta t} \ldots\ldots (10)$$

or:

$$Q = \frac{60(H_1 - H_2)A}{\Delta t} \quad \ldots\ldots\ldots (11)$$

The drawdown:

$$s = l + \frac{\Delta H}{2}\ ;\ \text{or}\ s = \frac{l_1 + l_2}{2} \quad \ldots (12)$$

A comparison of the previous equation for discharge (Eq 5) with Eq 11, shows that, instead of t, there is Δt. Instead of the calculated h, a definite figure for H is obtained. The new calculations eliminate a factor which would have required taking into account the cross-sectional area of the bucket, a. The whole problem of the volume of water contained between the well casing and the bucket can now be disregarded.

In wells without casing in the vicinity of the water table, after factor h has been removed, calculation of discharge becomes dependent on one factor only—the cross-sectional area of the well, A, or the diameter of the well, D—facilitating a more accurate calculation of the discharge. As already mentioned, in such a borehole there is no smooth wall surface, and it is not known if there are cavities in the neighborhood or whether stones have been removed from the walls during drilling. In such wells, it may be assumed that actual results are always likely to be better than those calculated, because A is always likely to be larger, although it is difficult to know just how much larger. The examples presented previously may be reconsidered, with different results.

TABLE 5

Calculations by Modified Method at Kfar Saba Well

Item	1st run	2nd run	3rd run
Measurements			
Lift of bucket (H)—*ft*	13.53	12.85	11.90
Time (t)—*sec*	5.00	4.50	3.95
Calculations			
Lift interval ($\Delta H = H_1 - H_2$)—*ft*	0.68	0.95	
Time interval ($\Delta t = t_1 - t_2$)—*sec*	0.50	0.55	
Discharge (Q)—*gpm*	783	986	
Bucket submersion after lifting (l)—*ft*	1.87	2.56	3.51
Drawdown (s*)—*ft*	2.23	3.05	

* $s = \frac{l_1 + l_2}{2}$.

TABLE 6

Calculations by Modified Method at Hittin Well

Item	1st run	2nd run	3rd run
Measurements			
Lift of bucket (H)—*ft*	9.95	9.05	8.55
Time (t)—*sec*	4.9	4.3	4.0
Calculations			
Lift interval ($\Delta H = H_1 - H_2$)—*ft*	0.90	0.50	
Time interval ($\Delta t = t_1 - t_2$)—*sec*	0.60	0.30	
Discharge (Q)—*gpm*	871	936	
Bucket submersion after lifting (l)—*ft*	5.15	6.06	6.56
Drawdown (s)*—*ft*	5.61	6.33	

* $s = \frac{l_1 + l_2}{2}$.

Kfar Saba Well

If the results of the calculations shown in Table 5 are compared with those of the actual pumping test given in Table 2, it is found that they are very close. Calculations indicated a yield of 986 gpm with a drawdown of 3.05 ft, as against 1,100 gpm during the pumping test with a drawdown of 3.38 ft. When the rates of yield, $\frac{Q}{s}$,

are compared, it is seen that the results are almost identical, 323 against 326 gpm per foot of drawdown. The previous conclusions of uniform discharge at different drawdowns are no longer valid. For each discharge there is now one corresponding drawdown figure.

It is to be emphasized that the principle of calculation on the basis of "intervals" was arrived at later in the study. The figures for H (0.68 ft and 0.95 ft) at the test of the Kfar Saba well were obtained unintentionally. To secure better and more accurate results, and to facilitate measurements, especially that of time, it is desirable to have bigger intervals—of about 2 ft. Also, great care must be taken in measuring the bucket lengths and reading the stop watch.

Hittin Well

As noted previously, the discharge during the actual pumping test at Hittin did not reach the rates indicated in Table 6. The peculiar features of uniform discharges at different drawdowns, however, have disappeared. The relative yield, $\frac{Q}{s}$, agrees well with the results of the test calculations, and its rate is higher when the drawdown is smaller.

The interval method of calculation has eliminated the initial-velocity influence and allowed an approach to an accurate calculation of the values of drawdown. These results show that the goal of measuring the yield of drilled wells without carrying out actual pumping tests has been achieved.

Design and Construction of Peters Dam, Marin County, California

William R. Seeger

A paper presented on Oct. 28, 1954, at the California Section Meeting, Long Beach, Calif., by William R. Seeger, Gen. Mgr. & Chief Engr., Marin Munic. Water Dist., San Rafael, Calif.

THE importance of water in the development of the United States has never been more forcibly brought out than in the growth of the West, with particular emphasis on California, whose unprecedented expansion and population increase is well known. It has become the responsibility of the water works men to increase the present and future supply in their communities by every available means, including maximum utilization of watersheds.

At this time, probably one of the most highly developed watershed areas in California is the Marin Municipal Water Dist. The ultimate development of its major watershed, 10 miles long with 16,000 acres, has recently been reached with completion of Peters Dam, one of a total of four. The development has been so complete and intense that the upper end of each lake lies within a few hundred feet of the downstream toe of the next higher dam on the creek.

Organized in 1912 under a legislative act of California, the district is located on the north side of San Francisco Bay. The service area, primarily a residential community with homes scattered over rolling hills, encompasses 136 sq miles. Marin County, like all other West Coast sections, is experiencing an unprecedented population growth, resulting in the need for continuous planning to increase the potential supply.

The district water supply is primarily furnished by five impounding reservoirs, four of which are located on one watershed. The latter are: [1] Lagunitas Dam, constructed in 1872, with a capacity of 0.127 bil gal and a water surface elevation of 791 ft; [2] Bon Tempe Dam, constructed in 1948, with a capacity of 1.3 bil gal and a water surface elevation of 717.5 ft; [3] Alpine Lake, constructed in 1918, raised in 1924 and 1941, with a present capacity of 3 bil gal and a water surface elevation of 646 ft; and [4] Peters Dam, constructed in 1953–54, with a capacity of 5.4 bil gal and a water surface elevation of 355 ft.

Site Location and Planning

Immediately after raising Alpine Dam for the first time, it became apparent to the district directors that additional water would have to be obtained in order to meet the demand. Therefore, they began negotiations for the purchase of watershed lands suitable for a future dam site. A condemnation action was brought for acquisition of the property, but its value, as set by the court, was considered too high to pay. It was not until 1946 that

the area was bought, at a price satisfactory to all concerned. In 1949, a general review of the entire district operation was made, along with the investigation of various facilities to determine their future adequacy. The result was a recommendation that an additional supply be developed immediately on Lagunitas Creek, which traversed the lands previously purchased. A comprehensive survey was undertaken by the district engineering department in order to ascertain the most economical and feasible place to locate a dam. The investigation was simplified somewhat by the fact that precipitation and runoff information was available from rainfall stations previously established at the three dams on the same watershed. Thus, the time-consuming element of stream gaging and rainfall and runoff studies was not necessary to the extent that is ordinarily required on the development of a new water supply. Just below the proposed site, in order to check the reports, a stream-gaging station was installed. It was kept in operation during the preliminary determinations, the design, and the construction.

Available records indicated that the watershed area was subject to certain climatological variances. It was anticipated that a period of a 7-year subnormal rainfall would occur once in every 50–60 years and that a 3-year subnormal cycle would occur every 20 years. Therefore, it became necessary not only to plan for the immediate requirements and future growth, but also to provide carryover storage to allow for a shortage in years of subnormal precipitation. The cyclic element is the important factor that has made it absolutely necessary and essential that all planning and development be worked out on a 20-year program, the policy of the district.

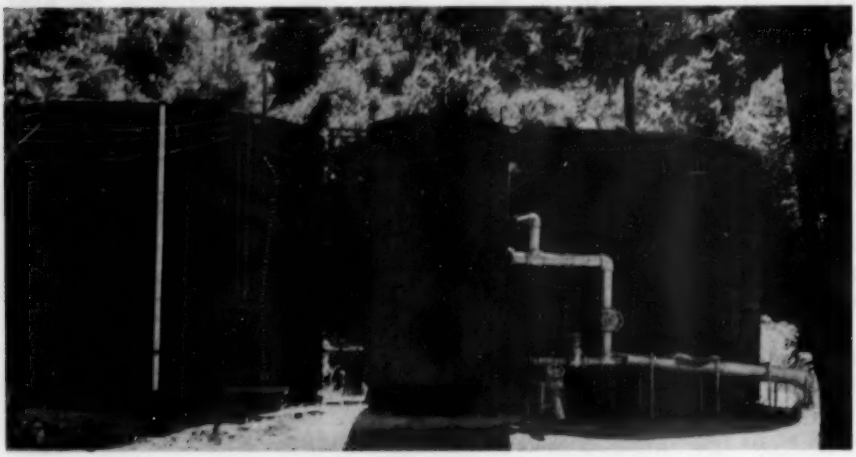

Fig. 1. Emergency Filter Plant

While the dam was under construction, this plant was used to supply water to the surrounding community.

From the past water consumption and population growth, a survey was made to be used as a basis of estimating the future increase in requirements. It was indicated that the community served would probably reach 116,000 by 1960, compared to 82,000 in 1948, and that the consumption would increase from a 1950 demand of 2.9 bgd to at least 5.4 bgd in 1970. In 1950 the total storage capacity was approximately 4.5 bil gal. Thus, the seriousness of the situation that motivated the immediate development of an additional supply is obvious.

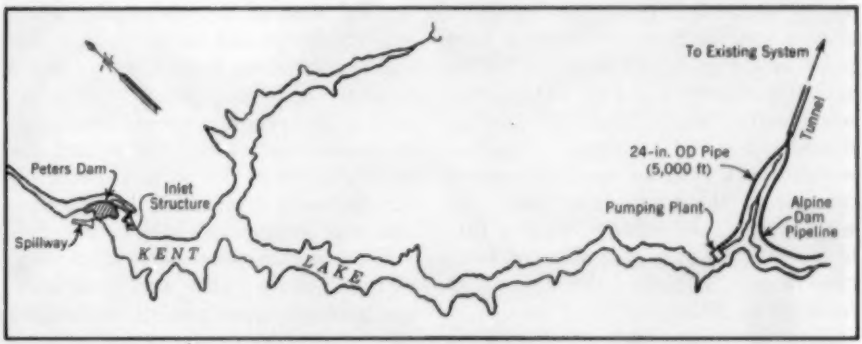

Fig. 2. Peters Dam Project

A temporary pumping plant and pipeline has been constructed to carry water from the upper part of Kent Lake to the main transmission system. In time, a permanent installation will be placed at the dam, from which a 9-mile pipeline will send water to the distribution system.

Preliminary Investigation

Aerial survey of watershed and dam site. The location of the proposed project was an area with an extremely heavy underbrush and a dense stand of timber, together with a very steep terrain. Surveying and reconnaissance of the section was quite difficult, so if the problem were handled by ordinary field survey crews, the cost would have been exorbitant. Therefore, in order to expedite the work and save money, an aerial map with 20-ft contours was prepared. Subsequent checks have shown the accuracy to be sufficient for use in determining reservoir capacity. The map was also used in selecting the approximate site of the dam.

Clearing of dam site and spillway area. From data supplied by the aerial survey, a height-capacity curve was developed, along with the tentative height of the dam. Because the wooded area made it impossible to fix the final location satisfactorily an area of about 40 acres was cleared of all vegetation at a bid of $535 per acre. A detailed topographic map of the area was then made by field crews and the final location was selected for the dam.

Emergency filter plant. A problem, unusual in the construction of a dam, developed from the clearing and burning operation: the falling timbers roiled up the stream flowing through the

canyon. The creek upon which the dam has been constructed is used as a source of supply for the surrounding community, making it necessary to furnish water all during construction. It was originally anticipated that an amount could be collected at night in two 50,000 gal tanks sufficient to serve the residents of the area during the working hours. Unfortunately, the scheduling of operations made this impracticable. There was not enough time for detailed study, plans, and specifications; arrangements had to be made immediately. The solution chosen was the construction of a filter plant (see Fig. 1). Primarily, the installation consisted of a 50,000 gal redwood tank, which provided mixing, flocculation, and settling. Another wooden tank held the sand and gravel comprising the rapid sand filter. A very satisfactory effluent with a turbidity below 5 ppm was obtained from raw water with turbidities ranging from 10 to 500 ppm.

Geologic investigation. Following the clearing of the dam site and spillway vicinity, a consulting geologist was employed to determine if the proposed site was satisfactory from a geologic standpoint. Excavations were made near crevices and in the area of the foundation and abutments.

This was done to ascertain the condition of the underlying strata, as well as to obtain an estimate of the amount of overburden to be removed. In all, about 12–14 excavations, done by heavy construction equipment, were made throughout the site.

The second phase of the investigation consisted of boring eight $2\frac{1}{8}$-in. diameter holes with diamond drills. These fair-sized cores were the largest obtainable without special equipment. The geologist thought the small extra cost required for these rather than smaller cores was worth while in the acquisition of foundation information. The holes were drilled along the path of a tentative cutoff trench, upstream from the proposed dam axis, which extended parallel to the face of the overflow weir of the spillway. The holes were drilled to depths ranging from 60 to 150 ft depending upon location, the deeper holes being nearer the bottom of the canyon than the shallower. Upon completion of the drilling, it was decided to obtain data on the tightness of the underlying strata to aid in design and construction. Several of the deeper holes were subjected to water tests and grouted. The results of this program proved invaluable in preparation of the design and the establishment of the grout pattern. It was definitely determined that the location was geologically satisfactory.

Soils investigation and laboratory. Simultaneously with the foundation and geologic investigations, an inquiry was undertaken into the probable depth, location, and amount of borrow material that would be available for use. About 62 12-in. auger holes, 8–45 ft deep, were drilled and sampled. Prior to this, a thorough study of the complete terrain had been made, with 25 excavations hand-augered to furnish specimens for compaction tests.

A fully equipped soils laboratory was set up and staffed by district personnel. Moisture density curve determinations were run on every hole. The results suggested that all materials to be encountered could be separated into three general types. One of each was sent to a commercial laboratory in order to check the findings on moisture density and to run shear and percolation tests. The tests indicated the typical soil would probably have a shear

angle between 33 and 37 deg with a corresponding cohesive strength of 800–1,000 psf. Therefore, a shear angle of 30 deg and a cohesive strength of 800 psi was selected for design purposes.

Design Features of Project

General description. The entire project (see Fig. 2) consists of: a combination earth and rock-fill dam; a chute spillway with an ogee weir entrance; an inclined outlet tower constructed along the bank of the reservoir; and, at the upper end of the lake, a pumping plant.

The terrain of the country in which the watershed is located more or less dictated the present method of taking water. Peters Dam is located on the same stream as Alpine Dam, the major source of supply, so that the upper extremity of the new lake is within 1 mile of the main transmission system running from the latter dam to the service area. If water were to be taken through the outlet tower located in the vicinity of the dam, a 9-mile route and a large pumping plant would be required. After a careful economic analysis, it was felt that a special 24-in. OD line, approximately 1-mile long from a pumping plant at the upper end of the lake (see Fig. 2) was justified. This is to be used for the next 5–10 years, but when the pumping costs become prohibitive and demand warrants, it would be feasible to build a new line, 9 miles long, from the dam and to construct there a major pumping installation to send water into the heart of the distribution system.

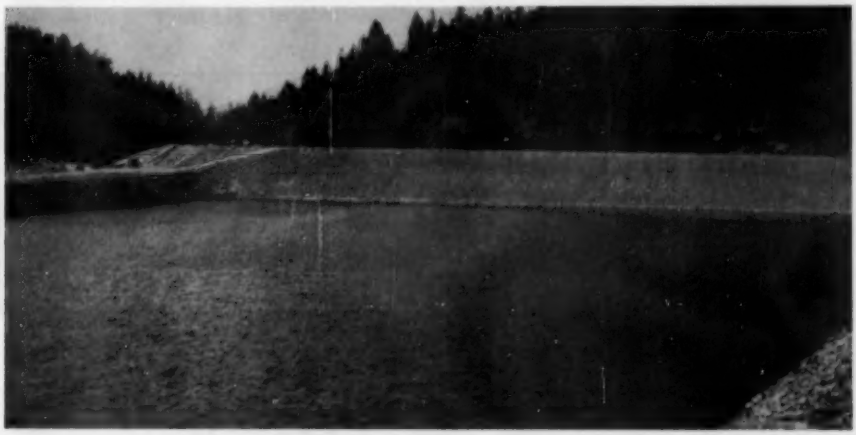

Fig. 3. Upstream Face of Dam

The overall height is 185 ft, with a crest width of 750 ft. Kent Lake is shown in the foreground.

Embankment design. The embankment consists of a central earth core and rock blankets upstream and downstream. Overall height of the dam is 185 ft, with a base length of approximately 1,100 ft and a crest width of 750 ft. Figure 3 shows the upstream face. The embankment was analyzed

by the Swedish slip-circle method to check for various conditions of drawdown, as well as to determine a horizontal earthquake allowance of one-tenth gravity. The most critical condition occurs with a sudden drawdown from a full reservoir to one-half capacity, the horizontal seismic-force allowance being equal to one-tenth gravity. The design slope finally selected was 2:1 on the upstream face of the impervious section and 1.5:1 and 2.5:1 on the downstream face, with an average of 3:1 on the upstream rock slope connects to a 36-in. cement-lined outlet pipe, encased in reinforced concrete, which runs through the dam, acting not only as a scour pipe, but also as a supply line. The concrete renders it capable of standing alone if the metal tube should deteriorate in the future. To prevent seepage, several cutoff collars were placed around the pipe extending into the abutment and embankment section. In order to permit selection of water from various depths of the lake, there are six 24-in. sluice gates installed at different elevations.

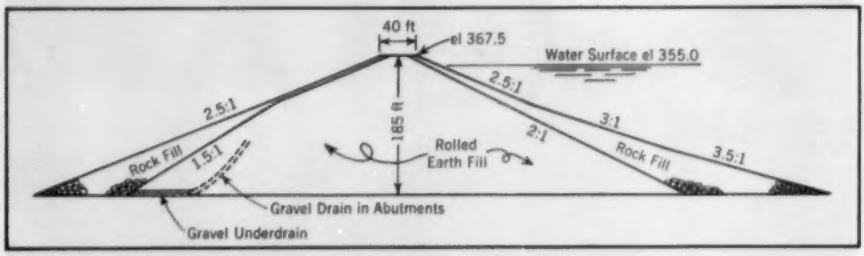

Fig. 4. Typical Cross Section of Dam

The dam is a combination of earth and rock fill. The drainage system permits the use of a steeper slope on the structure.

and 2½:1 on the downstream rock slope.

The plan also called for a gravel underdrain system with two vertical chimneys running up each abutment within the impervious section. The purpose of this arrangement was to lower the seepage line on the downstream face, allowing the use of a steeper slope. Figure 4 is a typical cross section of the structure.

Outlet works. The outlet works of the project consists of a 48-in. cement-lined steel pipe encased in a reinforced concrete jacket. This structure is installed as an inclined (1.4:1 slope) tower (see Fig. 5). The main tube These openings are operated manually by geared hoists mounted on pedestals at the top of the tower (see Fig. 6).

Spillway construction. The spillway excavation had a centerline cut, more than 200 ft deep, from which about 800,000 cu yd of material, primarily rock, was removed by drilling and blasting. When necessary, the crews worked three shifts, 7 hr a day, to keep ahead of the hauling equipment. Located on the left side of the dam, the spillway consists of a reinforced-concrete–lined chute channel with an ogee overflow weir. Its crest is 124 ft long, converging over a 200-ft section into a 30-ft trapezoidal channel

with 1½:1 slide slopes. The profile of the spillway is practically flat for the first 300 ft, runs downhill on a grade of about 40 per cent, and terminates with a bucket type of structure at the bottom, for dissipation of energy.

Pumping plant and transmission line. The plant located at the upper end of the lake, about 5½ miles from the dam, has a 5-mgd centrifugal pump powered by a 200-hp motor. The apparatus is mounted on a movable truck which can be lowered to the water level down a reinforced-concrete inclined ramp laid on a 1:1 slope (see Fig. 7). The water is taken from the top 35 ft of the lake. The supply is then chlorinated and discharged into the existing transmission system.

Construction of the Dam

Creek diversion. Because of downstream requirements for water, it was necessary that Lagunitas Creek continue to flow through the construction area at all times. The stream was diverted and sent against the left abutment, permitting workmen to proceed with the installation of the 36-in. outlet pipe adjacent to the right abutment. After this was completed, a coffer dam was placed 500 ft above the upstream toe of the dam. Then, water was diverted from the lake formed behind the embankment into the outlet pipe. Following this, the foundation area, now free of water, was cleaned of debris and all loose material.

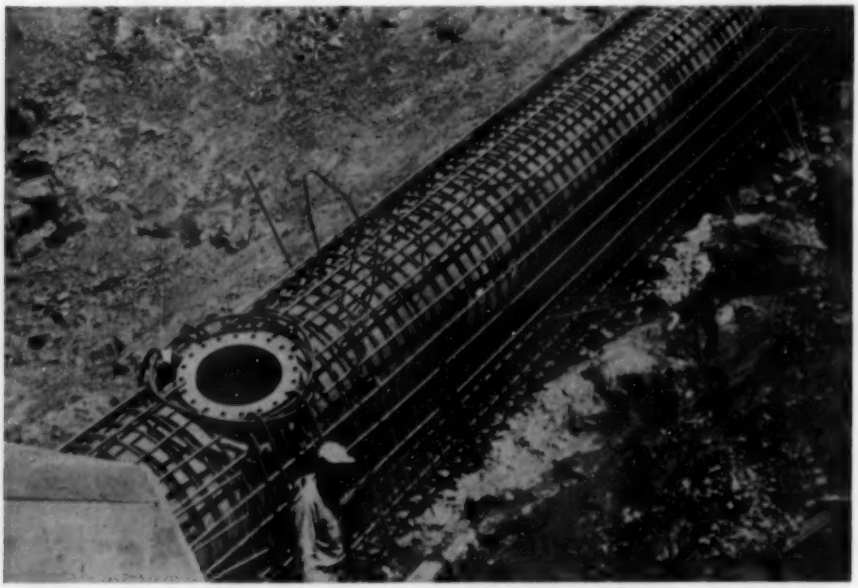

Fig. 5. Outlet Tower

The reinforcing steel is in place. At the lower left is an opening for a gate valve.

Foundation preparation. All overburden material was removed from the foundation. The overburden was removed from the abutments to a depth of about 10–15 ft above bedrock. Upstream along the axis of the dam was located a grout curtain the purpose of which was to provide a positive cutoff for any water that might find its way under the dam or through the foundation. The barrier was formed of about 177 holes, varying from 8 ft to 100 ft in depth. A total footage of 5,012 was drilled, with 1,697 cu ft of cement used in the grout. The first 10 ft was grouted, through which further drilling was done. Thus, increased pressures could be used at the greater depths without blowing out through the many small cracks existing in the foundation.

An unusual procedure was used in the preparation of the foundation because of the generally broken and shattered nature of the basalt rock encountered. The method was similar to that used by the US Army Corps of Engineers and the Bureau of Reclamation in dam construction. A grout wash was applied to the complete section upon which the impervious material and core was to be placed. The theory was that any small cracks that could possibly carry water would be packed by the mortar and prevent seepage from getting into and saturating the bottom portion of the fill. The grout blanket was placed either by brushing with ordinary brooms, or, as done in the latter part of the job, by gunite equipment.

Outlet pipe. Another unusual procedure was employed in the treatment of the excavation in connection with the installation of the outlet pipe (see Fig. 8). The basically solid material in which it was located airslacked very rapidly, so that continued exposure caused flaking and crumbling. One of the problems in the installation of outlet pipes is the final cleanup required prior to the placing of concrete, as well as the difficulty in making the cleanup when the pipe and reinforcing cage have been situated. These troubles were eliminated by the application of a coat of gunite to the floor and the abument along the pipe. This proved to be very satisfactory and economical; it allowed the placement of the pipe

Fig. 6. Operating Platform of Outlet Tower

The six geared hoists, operated manually, control the sluice gates.

and cage, with a minimum amount of dirt getting into the foundation. The final cleanup prior to the pouring of concrete consisted of an air-blowing and washing job that was done in a matter of minutes, but which ordinarily would have required days of hand mucking and removing of deleterious materials.

sluice gates had to be perfect so that they would not interfere with each other. The pipe and reinforcing cage was encased by concrete with a minimum thickness of 12 in. The procedure adopted was also rather unique. A set of inclined tracks was installed along one side of the outlet tower. The concrete was placed in a cart

Fig. 7. Pumping Plant

This movable apparatus can draw the top 35 ft of water from Kent Lake and send it to the main transmission line.

Outlet tower. Fastened to the rock foundation by means of anchor bars spaced 10 ft apart, center to center, the 48-in. pipe, set on saddles and welded in place, was put in position by means of a specially constructed "high line." Very close control was required for the installation of the pipe as it had a definite slope requirement. Moreover, the alignment of the outlet ports for the

which was sent up and down the slope by a cable operated from the top of the structure. After the pouring, the sluice gates, the screens over each gate, and the guide rods were installed, along with the pedestals at the top of the tower.

Placement of embankment. Material for the embankment was obtained within a 1,500-ft radius of the dam,

but a difference in elevation between the site and the diggings amounted to as much as 350 ft vertically. The spillway, with a maximum cut of 200 ft, constituting the major borrow area, was the sole source of rock. The remaining earth fill was obtained from the surrounding hillside, excavation depths ranging from 30 to 50 ft.

Record speed was made in the placement of the embankment: the 936,000 cu yd of material was placed in only 17 weeks, with a maximum weekly rate of 89,000 cu yd. In order to accomplish this feat, the crews worked two

Fig. 8. Outlet Pipe

The concrete saddles and reinforcing steel are in place.

10-hr shifts, 6 days a week for earth moving and 7 days a week for rock placement. For night work, an extensive and highly efficient lighting system was installed, greatly adding to the speed of completion. Compaction was accomplished by means of four rollers, loaded with sand and water. Moisture content was maintained on the fill by sprinkling from water wagons as it was impossible to "wet down" in the borrow areas. An unusual method of compaction against the irregular abutments was the use of a fully loaded rubber-tire scrapper unit, which eliminated a considerable amount of hand tamping. The efficiency of this was borne out by tests.

Embankment control. The design criteria called for a compactive effort equivalent to 21,600 ft-lb of energy per cubic foot of material. Control of the compaction was obtained by taking frequent samples (a total of 235) and comparing the density against that of a theoretical laboratory test. The volume of the specimen was determined by filling the hole from which it came with a known quantity of sand. The laboratory sample was prepared by placing in a $\frac{1}{30}$-cu ft cylindrical mold three layers of earth, each of which was tamped with sixteen blows of a 10-lb hammer dropped 18 in. Thus, 21,600 ft-lb of compactive effort was applied per cubic foot of soil.

At times, considerable irregularity was encountered in the comparison of the theoretical preparation with field samples containing an excessive amount of material over $\frac{1}{4}$ in. in diameter. The difficulty was not unusual, as it seems that there has been a lack of research on the problem, although apparently it has been encountered elsewhere. Whenever the sample contained 20–25 per cent of material greater than $\frac{1}{4}$ in. in diameter, the standard comparison seemed to lose some of its usefulness. The best alternative was to rely on a combination of laboratory results and visual observation.

The methyl alcohol method * was used to determine the soil moisture content, making it possible to obtain the results within two hours of fill

* This consists, basically, of adding methyl-alcohol to a small soil sample and filtering the mixture. By obtaining the temperature and specific gravity of the filtrate, it is possible to ascertain the moisture content.

placement. Oven-dried samples were also run daily as a check, which showed that the methyl alcohol method was consistently within 2 per cent of the oven-drying procedure. The fill, as placed, was generally up to standard.

Spillway construction. The 40 per cent grade of the spillway presented a very complicated problem because special equipment and gear was necessary in the pouring. The design had provided for meeting uplift stress by setting anchor bars in the rock on 5-ft centers. During construction, the material encountered was somewhat different than had been anticipated, as well as being considerably broken and cracked in many locations. Therefore, special drains were placed to relieve any possible uplift stress that might be caused by springs in the spillway walls and bottom.

Where roadways were easily accessible, the concrete was placed by means of a crane. In other sections, a single-displacement piston pump discharged concrete into a 7-in. diameter pipeline, whose capacity of placement was approximately 15 cu yd per hour. This method * was very satisfactory, little difficulty being encountered.

* The "Pumpcrete" method of Chain Belt Co., Milwaukee, was used.

Survey work. Survey control for the project was by means of imaginary 50-ft grids over the entire area. They were indicated by red and white targets set on nearby hillsides. Three survey crews were kept busy 6 days a week in order to set line and grade, as well as to obtain information for progress payments. During the peak of construction, the engineering staff of the district consisted of 24 men, including survey men, inspectors, soil testers, and office personnel. An office was erected at the site of the job and all work was directed from there.

Acknowledgment

The personnel of the Marin Municipal Water Dist. was under the overall command of James S. Peters, Gen. Mgr. and Chief Engr. The design and construction of the project were under the direction of the author. The consultant was E. E. Blackie, of Blackie and Wood, Civ. Engrs., San Francisco. The resident engineer in charge of construction for the district was H. K. Brainerd, with William Grant as project engineer. Chief engineer for the contractor was R. H. Heitman, with Earl Bernard acting as general superintendent for the John Delphia Co.

Use of Permanent Standards in Fluoride Analysis

R. E. Frazier

A paper presented on Oct. 7, 1954, at the Minnesota Section Meeting, St. Paul, Minn., by R. E. Frazier, Chief, Eng. Labs. Sec., Div. of Environmental Sanitation, State Dept. of Health, Minneapolis, Minn.

IN fluoridating relatively small municipal water supplies it is highly desirable to have some means of analytical control of the operation where laboratory services are not readily available. The method of control must be one that approaches a field test in simplicity and ease of operation. It is possible to set up a rather simple system in which comparison standards are made up of fluoride solutions each time the analysis is made. Permanent standards, however, are even simpler, and commercial kits which employ either permanent colored-glass standards or permanent liquid standards are available.

The use of fixed standards of one type or another in colorimetric analysis probably developed almost as soon as the idea of colorimetric analysis itself. The production of standards, whether of colored-glass or colored-chemical solutions, was largely a trial-and-error method. The accuracy of the visual matches obtained was certainly influenced in part by any peculiarities in vision of those who made the standards. Today, spectral analysis of colored material is so well developed that color standards can be made which match the colors developed in actual analysis very closely in all respects. Any problems in the use of permanent color standards do not arise from inability to make good standards. Difficulties are the result of other factors in the system of analysis.

With both the Hellige * and Taylor † comparators, the color developed in the presence of fluoride ions depends on the action of the fluoride in decolorizing the red lake formed by zirconium and alizarin in a solution of a certain acidity. This is also the basis of the most commonly used laboratory system of analysis for fluoride in water. The major difference is in the use of permanent color standards.

Analysis Interferences

Certain substances may be present in potable water in sufficient concentration to cause difficulty in fluoride analysis. Color and turbidity give trouble as well. Correction for the interfering substances may be made in several ways. The most reliable one is to distill the fluoride away from the rest of the sample. This, of course, is not a practical field method. Simpler corrections can be made arithmetically if the degree of interference is constant and known. Chlorine may be de-

* Hellige, Inc., Long Island City, N. Y.
† W. A. Taylor & Co., Baltimore.

stroyed by thiosulfate, arsenite, or sulfite. In many cases, especially with fluoridated water, it is possible to dilute the sample with distilled water until the interfering substance is of so low a concentration that the interference is not significant.

Interferences of this type affect laboratory methods as well as field methods. In general, corrections are made in the same manner for both types, and the compensation for color and turbidity is more convenient with the field apparatus. Accurate measurement of sample and indicator is required in both cases, and clean equipment is essential for all good colorimetric work.

The use of polyphosphate in stabilizing water has presented some difficulties in fluoride analysis in the past. The amount of polyphosphate which will cause interference in the test is small and the addition of polyphosphate has not always been a closely controlled operation. For the past several years, however, determinations on distilled and undistilled samples for fluorides in polyphosphate-treated water in Minnesota have not indicated an excess of polyphosphate. Distillation is rarely necessary to correct for errors in fluoride reading caused by this type of treatment. The use of alum or aluminate in certain surface water treatment plants appears to cause some trouble, which should be corrected by distillation.

Use of Field Kits

There are several specific precautions to be observed in using field kits. The quality of the indicator reagent must be unvarying. The manufacturer's stipulation of the length of time the reagent is stable and his instructions on storage should be noted and followed. In addition, care should be taken to avoid contamination of the reagent.

Further, the temperature at which the test is made should conform to manufacturer's requirements. The sample should be adjusted to the proper temperature, using an accurate thermometer for the measurements. The place where the comparator is kept for the hour waiting period should also be at the correct temperature.

The time between the addition of the reagent and the comparison of the color developed must conform to the directions for the instrument used. More than a minute or two variation can affect the reading. It is also good practice to dilute the sample, if necessary, to avoid readings at the extreme upper limit of the instrument.

The US Public Health Service feels that comparators using permanent standards can give results correct within 0.2 ppm fluoride, if used properly. The limited data obtained by Minnesota Dept. of Health laboratory checks of samples submitted by various water plant operators against determinations made in the field by the operator indicates that most results are correct within 0.1 ppm. This is certainly sufficiently accurate for control purposes.

Conclusion

Experience to date seems to indicate that permanent standards can be used wherever the direct colorimetric analysis can be applied. Also, when distillation is necessary, the permanent standards can be used in analyzing the distillate. Results obtained with the commercial kits, supplemented by data obtained from records of water pumped and pounds of fluoride fed, should give all information necessary for control of most fluoridation installations.

Discussion

Harry P. Kramer

Chief, Training, Robert A. Taft San. Eng. Center, Cincinnati, Ohio.

There can be no question that the use of permanent standards for visual comparison offers certain definite advantages. Equipment needs are standardized, simple, and relatively inexpensive. The manipulation required for a specific determination is lessened since the rather laborious preparation of temporary standards is made unnecessary. Usually this permits a larger number of determinations to be performed in the same time interval.

Certain disadvantages in the use of permanent standards must also be taken into account. Even in the cases where a spectral match is made between reagents and standards, and the problem of different hues produced by varying sources of illumination, as well as different depths of color, has been reduced to a minimum, there remains the question of the quality of the specific reagent batch.

Fluoride reagents made up from different stocks of alizarin and zirconium can and do exhibit different color intensities. A check against the permanent standards should be made each time a new stock of reagent is prepared. Furthermore, Frazier made no mention of the different reagents required for use with the Hellige and Taylor systems, one based upon the Scott-Sanchis and the other the Lamar reagent.

Areas of disagreement exist about the rigidity of the time requirements in the use of permanent standards and the accuracy claimed in Frazier's paper. There is no lessening of the requirements for accurate measurement of reagent when using permanent standards, and this is a most critical step in the fluoride procedures.

As stated by the author, the experience of the US Public Health Service with permanent standards has indicated that results within 0.2 ppm fluoride are possible. The writer's experience has also shown that the use of the Nessler tube technique yields results within 0.1 ppm. The author indicates that upon limited data he has observed results within 0.1 ppm with permanent standards.

These are two reasons why it would actually be difficult to stay consistently within that degree of accuracy. First, there is a great advantage in reading a long tube of color when done visually, as done with the Nessler tubes, over reading a short tube, as usually done with the permanent standards. Second, any constant errors in the measurement of reagents would be common to both standards and sample when using the Nessler tube technique.

EDITOR'S NOTE: AWWA Committee E5.4—Fluoridation Materials and Methods is now developing a testing program to determine the sensitivity and accuracy of proprietary fluoride test kits under conditions of recommended use, as well as under other conditions.

Factors Affecting the Determination of Fluoride in Water With Zirconium-Alizarin

William L. Lamar and Paul G. Drake

A contribution to the Journal by William L. Lamar, Dist. Chemist, and Paul G. Drake, Chemist, both from US Geological Survey, Columbus, Ohio. Publication authorized by the Director, US Geological Survey.

THE determination of fluoride by the zirconium-alizarin reaction has been studied by many investigators, beginning with De Boer. Among methods presently in general use are those of Sanchis (*1*), as modified by Scott (*2*), Lamar (*3*), and Megregian and Maier (*4*). These are reported in *Standard Methods* (*5*).

In the Sanchis procedure, the reagents of hydrochloric acid, sulfuric acid, and zirconium-alizarin were added separately. In the Scott modification, these solutions were mixed and added together, improving the technique. It was found, however, that the mixed-acid indicator was rather unstable. The Lamar procedure provided a stable indicator by acidifying with only sulfuric acid. Megregian and Maier proposed a photometric procedure using the mixed acids. Also, in view of the aging characteristics of the mixed-acid indicator, they recommended that the alizarin be added separately.

The determination of fluoride with the zirconium-alizarin reagent is based on the ability of the fluoride ion to form the zirconium fluoride complex ZrF_6^{--}. This results in proportionate subtraction of zirconium from the zirconium-alizarin indicator. It varies, when fluoride is present, in color intensity, ranging from pink to yellow-green, with intermediate combinations in quantitative proportion to the concentration of the fluoride ion. As the color approaches yellow-green, the test is less sensitive.

Zirconium-alizarin is used rather extensively for the determination of fluoride in water, which is done readily by the addition of a single reagent. This method, as well as others, is subject, however, to interferences from various substances. Small to large errors may ensue, depending upon the effect of one or more of the interferences. For the readings made at 1 hr, reaction timing and temperature differentials may also be a source of considerable error. This may be negligible, however, when the reaction time is sufficient to approach equilibrium conditions (over 7 hr).

In using the zirconium-alizarin reagent, it is very important to understand and give particular attention to the principles involved. This paper indicates how and when the procedure may be used to obtain the desired accuracy. The sources of error and conditions of the reaction are specifically de-

fined. Data are provided on temperature, time of reaction, interferences, conditions under which corrections may be applied, and isolation of the fluoride by distillation. Also, improvements in technique are presented for both the direct and distillation procedures.

Reagents and Preparation

All reagents should conform to specifications of the Committee on Analytical Reagents of the American Chemical Society (*6*). When the specifications are not known, the chemicals used must be sufficiently pure to obtain the required accuracy. The acid indicator and alizarin should be stored in the dark, the fluoride standard being kept in resistant glass or polyethylene.

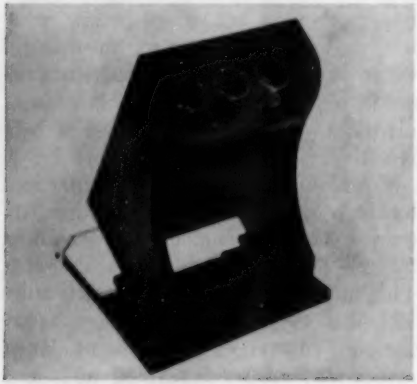

Fig. 1. Three-Hole Comparator

Readings for fluoride should be made in north or fluorescent light.

Zirconyl nitrate or zirconyl chloride. Dissolve 1.84 g $ZrO(NO_3)_2 \cdot 2H_2O$ or 2.22 g $ZrOCl_2 \cdot 8H_2O$ in 250 ml distilled water. If a turbid solution results, it should not be filtered. When pure chemicals are used, a clear acid indicator will develop on mixing the reagents.

Alizarin red S (certified). Dissolve 0.37 g alizarin monosodium sulfonate in 250 ml distilled water.

Sulfuric acid, 2.1N.

Acid indicator. Add 25 ml of the zirconyl solution to 50–100 ml distilled water. With constant stirring, slowly add 25 ml of the alizarin solution. Make up to 500 ml with distilled water. Mix well and add 500 ml of 2.1*N* sulfuric acid. The acid indicator, ready for use in about 1 hr, should be clear. This substance is quite stable, particularly when stored in the dark, so that refrigeration is not necessary. If a precipitate forms or a poor color range develops, a fresh reagent should be prepared.

Sodium arsenite (meta). Dissolve 0.916 g $NaAsO_2$ in distilled water and dilute to 500 ml.

Sodium hydroxide, approximately 0.5N. Dissolve 5 g NaOH in distilled water and make up to 250 ml.

Sodium fluoride. Stock solution: dissolve 0.2210 g sodium fluoride in fluoride-free distilled water and make up to 1 liter. Standard solution: dilute 100 ml stock solution to 1 liter with fluoride-free distilled water, so that 1 ml contains 0.01 mg fluoride.

Fluoride-free distilled water.

Procedure

Transfer 100 ml, or a volume containing not more than 0.14 mg fluoride, of the clear samples to Nessler tubes. If the volume of the sample taken is less, make up to the full quantity with distilled water. Although concentrations of the fluoride ion as high as 0.14 mg can be compared, the accuracy is improved if the amount of fluoride is limited to not more than 0.10 mg. In

water supplies containing 1.0–1.2 ppm fluoride, it is more satisfactory to use a 50-ml sample diluted to 100 ml with distilled water. This will provide a color comparison in the most sensitive area and reduce the concentration of interfering ions. Matched Nessler tubes, preferably the short form, should be used throughout the procedure. If chlorine is present, it should be removed by adding a slight excess of the sodium arsenite solution, 0.1 ml more than 0.1 ml for each 0.1 mg chlorine in the sample aliquot.

Prepare a series of standards in intervals of 0.01 mg fluoride, covering the anticipated range of the samples. Make the standards up to 100 ml with distilled water. To each of the samples and standards, add exactly 10 ml of the zirconium-alizarin reagent and mix well. The height of the liquid should be the same, or practically so, in sample and standard, or errors will result. Compare the samples to the standards in a three-hole comparator (see Fig. 1), using north light or illuminating the instrument by fluorescent light.

The readings may be made at the end of a 60-min reaction time if the reagent was added to all the tubes within 2 min, provided both test elements are maintained at the same temperature. The time allowance is made on basis of a maximum permissible difference in color comparison equivalent to 0.005 mg fluoride. If the results are not needed promptly, the most satisfactory procedure is to make the comparisons after a reaction time of 7–18 hr, when color range is better. Moreover, the temperature and the time used in adding the reagent are not critical. When the reaction time is not less than 7 hr, differences up to 1 hr in the ages of the samples and standards are permissible. Inspection of the samples will indicate any concentration of fluoride requiring a smaller aliquot, which may be taken and prepared within the 1 hr limit.

Interferences

Some of the ions commonly found in natural and treated waters affect the reaction in such a manner that the fluoride concentration appears to be higher than its true value, other ions having the opposite effect. The interference of appreciable quantities of calcium (200 ppm), magnesium (200 ppm), sodium (1,000 ppm), potassium (200 ppm), and nitrate (300 ppm) is negligible. In general, higher concentrations of these ions will cause very little trouble. In concentrated samples, however, there will generally be appreciable interference from other ions, so that isolation of the fluoride by distillation is usually advisable.

When the summation of the interferences does not exceed the equivalent of 0.1 ppm fluoride, no corrections are necessary, as this is about the accuracy of routine analysis. Although good results can be obtained by the application of limited corrections, isolation of the fluoride will provide the maximum accuracy when the total interferences exceed the equivalent of 0.1 ppm fluoride. Moreover, when the composition of the water is not known, distillation is required.

The aluminum ion causes a negative error, because of the formation of an aluminum-fluoride complex which limits the availability of the fluoride ion for formation of the ZrF_6^{--} ion. The magnitude of the difficulty is dependent on the amount of fluoride and aluminum present. The percentage of the

fluoride complexed by the aluminum was found to be constant, for a given concentration of the latter, within the limits of experimental error. The ratios of actual to apparent (observed) fluoride are shown in Table 1. These are valid for a reaction time of 1 hr and a period of 7–18 hr.

TABLE 1

Effect of Aluminum Interference

Aluminum Added *mg/100 ml*	Fluoride Added *mg/100 ml*	Fluoride Observed *mg/100 ml*	Ratio of Actual to Apparent Fluoride
0.02	0.050	0.045	
	0.070	0.065	1.08
	0.100	0.095	
0.05	0.020	0.017	
	0.050	0.044	
	0.070	0.061	1.14
	0.100	0.090	
0.10	0.050	0.040	
	0.070	0.055	1.24
	0.100	0.083	
0.16	0.050	0.037	
	0.100	0.075	1.35
0.20	0.050	0.035	
	0.070	0.050	1.41
	0.100	0.072	
0.25	0.020	0.013	
	0.050	0.035	1.48
0.30	0.050	0.032	
	0.070	0.044	1.56
	0.100	0.066	
0.40	0.050	0.030	
	0.070	0.042	1.67
	0.100	0.060	
0.50	0.050	0.027	
	0.070	0.038	1.83
	0.100	0.056	
0.60	0.050	0.025	
	0.070	0.035	2.00
	0.100	0.050	
0.80	0.050	0.020	
	0.100	0.045	2.32
1.00	0.050	0.017	
	0.100	0.038	2.75

It is possible to correct the fluoride reading for interference due to aluminum. A plot (see Fig. 2) of the ratio of actual to apparent fluoride against aluminum provides a convenient means for obtaining the correction factor. For the concentration of aluminum in the sample aliquot, the ratio of actual to apparent fluoride may be taken from Fig. 2. This factor when multiplied by the apparent fluoride will give the actual. Before correcting for aluminum interference, corrections for the other substances must have been made. Very small amounts of aluminum have a varying effect on the fluoride measurement depending upon the concentration of fluoride. The presence of 0.02 mg aluminum in the sample aliquot causes a negative error equivalent to 0.005 mg fluoride. For higher aluminum concentrations, the error varies according to the fluoride concentration in the sample aliquot. For fluoride concentrations not exceeding 0.10 mg and aluminum concentrations not exceeding 0.05 mg, the maximum of the variable errors will not exceed the equivalent of − 0.01 mg fluoride.

The presence of hexavalent chromium causes a positive error in the fluoride determination. This effect varies inversely with the concentration of fluoride and directly with the concentration of chromium and the reaction time (see Table 2). When the chromium in the sample aliquot exceeds 0.01 mg, the fluoride readings should be taken at 1 hr; when the chromium exceeds 0.05–0.06 mg, the fluoride should be isolated by distillation.

Ferrous and ferric iron up to 1.5 mg in the sample do not cause any significant amount of interference, but 2.5 mg of iron interferes seriously, as the zirconium-alizarin lake and free alizarin colors are obscured (see Table 3). Manganese does not produce any significant interferences in quantities as high as 2.5 mg in the sample (see Table 3).

The presence of sulfate increases the fluoride reading, while chloride and alkalinity decrease it (see Table 3). These effects in the sample are substantially additive, and corrections may be applied up to 100 mg sulfate, 200 mg chloride, and 50 mg alkalinity as $CaCO_3$. There is no difference in the effect of bicarbonate, carbonate, and hydroxide alkalinity. When a sample of less than 100 ml is taken, the alkalinity of the diluted sample must be used.

The interference of nitrate found in natural and potable waters is negligible (see Table 3).

Phosphates as PO_4 or as $(NaPO_3)_6$ are similar in their effects, decreasing or increasing the fluoride reading according to the quantities of phosphate and fluoride present, and varying with the reaction time. It is possible that there may be simultaneous complexing of fluoride with phosphate, decreasing the apparent fluoride, and formation of secondary zirconium phosphate, increasing the apparent fluoride. If these reactions proceed at different rates they may be the cause of the variable interference.

When the concentration of fluoride in the sample aliquot is not more than 0.10 mg, the maximum of the variable interferences of phosphate as PO_4 is as follows: [1] for 1 hr readings when the phosphate does not exceed 0.10 mg, the error is not more than 0.004 mg fluoride; [2] for 1 hr readings when the phosphate does not exceed 0.20 mg, the error is not more than 0.007 mg fluoride; and [3] for 7–18 hr readings when the phosphate does not exceed 0.20 mg, the error is not more than 0.004 mg fluoride. If the concentration of phosphate in the sample aliquot exceeds these limits, the fluoride should be isolated by distillation. For the ranges of fluoride and phosphate given, the errors are mostly negative. For several combinations of the higher concentrations of fluoride and the lower concentrations of phosphate, however, the error is positive, the maximum positive occurring for the combination of 0.10 mg fluoride and 0.01 mg phosphate.

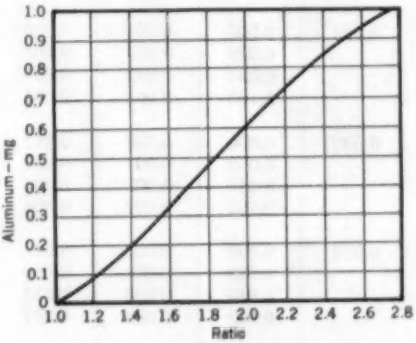

Fig. 2. Ratio of Actual to Apparent Fluoride

The correction factor for aluminum interference may be secured from this graph.

Shown in Table 4, the effect of acidity, increasing the fluoride reading, became observable at and below pH 4.0. At this point, the effect is equivalent to only + 0.02 mg fluoride. Although the hydrogen ion effect may be counteracted by applying corrections or by bringing the pH of the sample to between 4.5 and 6.0 by the careful addi-

tion of sodium hydroxide, waters low in pH are likely to have other constituents present which would make the isolation of the fluoride by distillation advisable. When sodium hydroxide is used to raise the pH, however, a 0.5 *N* solution is called for to keep the volume added to a minimum, as the amount and height of liquid in sample and standard should be nearly the same. As pH approaches 4.5, care must be taken with the NaOH not to overrun the pH range of 4.5 to 6.0, nor to go above 7.0. A weaker NaOH solution may be used when the liquid in the test elements is adjusted to the same quantity. When less than a 100-ml sample is taken and made up to the desired measure, the pH of the diluted specimen may be used.

TABLE 2

Effect of Hexavalent Chromium Interference

Chromium Added *mg/100 ml*	Fluoride Added *mg/100 ml*	Fluoride Error (Positive) *mg/100 ml*	
		1 Hr	7–18 Hr
0.010	0.000	0.005	0.005
	0.020	0.004	0.004
	0.050	0.002	0.003
	0.100	0.001	0.002
0.020	0.000	0.005	0.009
	0.020	0.003	0.007
	0.050	0.003	0.006
	0.100	0.002	0.005
0.030	0.000	0.005	0.012
	0.020	0.003	0.010
	0.050	0.003	0.006
	0.100	0.002	0.004
0.050	0.000	0.005	0.022
	0.020	0.003	0.016
	0.050	0.003	0.012
	0.100	0.002	0.007
0.060	0.000	0.007	—
	0.020	0.007	—
	0.050	0.003	—
	0.100	0.002	—
0.080	0.000	0.012	—
	0.020	0.012	—
	0.050	0.007	—
	0.100	0.006	—
0.100	0.000	0.015	—
	0.020	0.015	—
	0.050	0.010	—
	0.100	0.008	—

The comparison of the samples is obscured by turbidity and color of the water. Turbid samples should be filtered clear. The effect of color may be compensated for to a limited extent. Place the sample above a Nessler tube containing distilled water; place the standard above a duplicate sample which has been acidified with 5 ml of 2.1*N* sulfuric acid. Each tube should be made up to contain equal volumes.

Isolation of Fluoride

When interfering substances are present in significant quantities, it is necessary to provide for their removal prior to the fluoride determination. This may be accomplished by a suitable distillation of the sample, a procedure described in following paragraphs. The fluoride is microdistilled as hydrofluosilicic acid. Preliminary concentration of the sample is necessary to promote complete recovery.

Apparatus. The device shown in Fig. 3 consists of a steam-generating assembly, a fluoride distillation section, and a distillate-receiving flask. Rubber connections are used on the steam-generating assembly, while ground-glass joints are used on the fluoride distillation section. This was modified from a commercial fluoride distillation

apparatus.* The hot water reservoir (*B*) supplies boiling water to the steam generator (*E*). The pressure control tube (*A*) serves as a steam release. When boiling water is needed in the steam generator (*E*), the clamp (*D*) is released and air pressure is applied to the tube (*A*). Before each use the distillation section is washed with a hot 10 per cent sodium hydroxide solution and water. Following this, sulfuric acid is heated to fuming in the flask, followed by cooling and thorough rinsing with water.

Reagents. These are: [1] sodium hydroxide pellets; [2] sulfuric acid, concentrated; [3] silver sulfate powder; and [4] phenolphthalein indicator.

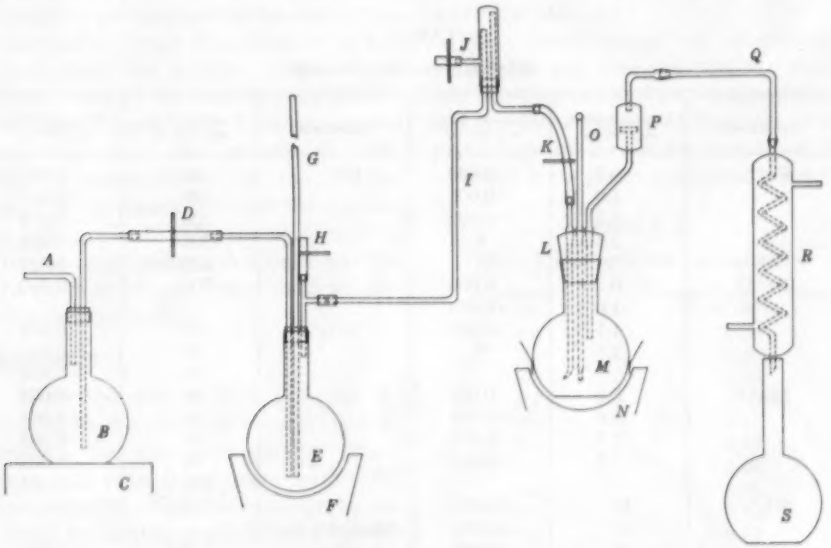

Fig. 3. Distillation Assembly for Fluoride Isolation

A—Pressure control tube. B—Hot-water reservoir, 1,000 ml. C—Hot plate. D—Hot-water supply tube clamp. E—Steam generator flask, 1,000 ml. F—Heating mantle. G—Pressure relief tube. H—Steam regulation tube, with Hoffman clamp. I—Glass steam line (asbestos covered). J—Steam trap (asbestos covered), with clamp on relief water drain. K—Steam inlet tube, with Hoffman clamp. L—Fluoride distillation flask adapter. M—Fluoride distillation flask, 300 ml. N—Heating mantle with asbestos shield. O—Thermometer, 250°C. P—Trap. Q—Condenser adapter. R—Condenser. S—Distillate-receiving flask, 200 ml.

Distillation procedure. Transfer 200 ml of the sample to the fluoride distillation flask, make alkaline to phenolphthalein with a pellet of sodium hydroxide, add some glass beads, and evaporate to a volume of 15–20 ml, preferably using an electric heating mantle. Care must be exercised to

* Ace Glass No. 6430, a product of Ace Glass, Inc., Vineland, N.J.

prevent bumping which will result in loss of sample. Allow the concentrated sample to cool, make slightly acid by the dropwise addition of concentrated sulfuric acid, and add sufficient silver sulfate (a slight excess will do no harm) to precipitate any chloride.

TABLE 3

Effect of Various Substances

Substance	Concentration mg/100 ml	Fluoride Error mg/100 ml	Substance	Concentration mg/100 ml	Fluoride Error mg/100 ml
Fe^{++}	0.2	0.000	Cl^-	60	−0.007
	1.0	−0.001		100	−0.012
	1.5	−0.002		200	−0.018
	2.5	*		300	−0.023
Fe^{+++}	0.2	0.000		400	−0.025
	1.0	−0.001		500	−0.030
	1.5	−0.002	NO_2^-	10	−0.002
	2.5	*		20	−0.002
Mn^{++}	0.2	0.000		30	−0.002
	1.0	−0.001		40	−0.004
	1.5	−0.001		50	−0.005
	2.5	0.000		60	−0.005
SO_4^{--}	10	+0.002		80	−0.006
	20	+0.002		100	−0.008
	30	+0.003	Alkalinity as $CaCO_3$		
	40	+0.006		2.5	−0.002
	50	+0.009		5.0	−0.002
	60	+0.011		7.5	−0.003
	100	+0.018		10.0	−0.005
	140	+0.029		12.5	−0.006
	180	+0.037		15.0	−0.007
	200	+0.038		17.5	−0.009
Cl^-	20	−0.002		20.0	−0.011
	30	−0.003		25.0	−0.013
	40	−0.004		30.0	−0.017
	50	−0.006		40.0	−0.023
				50.0	−0.028

* Obscured by color from the iron.

Connect the distillation assembly, except for the steam inlet tube (*K*). In place of this, insert a long-stemmed cylindrical separatory funnel. Start a flow of cooling water through the condenser, close the stopcock on the funnel, and introduce 20 ml of concentrated sulfuric acid into it. Slowly add the acid to the fluoride distillation flask, regulating the flow of acid by means of the stopcock. Remove the funnel (wash the tip of funnel into flask). In its place, insert the steam inlet tube, connecting it to the steam trap (*J*).

Start the flow of steam through the sample by opening the clamp on the steam inlet tube (*K*) and by closing the clamp on the steam regulation tube (*H*). Adjust the clamp so that a hydrostatic head of 20–40 cm is maintained in the steam generator pressure

relief tube (*G*). The distilled water in the steam generator should be made slightly alkaline to phenolphthalein with NaOH.

Apply heat to the distillation flask with the heating mantle (*N*). Use a shield of asbestos at the top of the mantle to prevent undue heating of the flask walls. Heat should never be applied above the surface of the liquid. Collect enough distillate to fill a 200-ml volumetric flask while maintaining the temperature of the sample at 135–145°C, accomplished by the adjustment of the steam flow and the amount of heat applied. The distillate will be free of interfering substances so that the fluoride may be determined as previously described.

Summary

With the proper technique and attention to the effect of interfering substances, the zirconium-alizarin procedure will provide accuracy within 0.1 ppm fluoride. Good results can be obtained by limited application of corrections. When the interferences are noticeable, however, maximum accuracy is obtained by isolation of the fluoride by distillation. This is also necessary for samples of unknown composition. Readings at 1 hr have the advantage of making the results available more promptly, but are a source of error if reaction time and temperature controls are not precise. These factors are not critical for readings taken at 7–18 hr.

The effect of interferences is greater than has been reported by other researchers. The extent of the effect, however, varies with the concentration and kind of acid or acids used in the preparation of the zirconium-alizarin reagent.

An advantage of the sulfuric acid indicator is its practically indefinite stability. In contrast, the hydrochloric-sulfuric acid indicator is not predictably stable. In some instances, precipitation of the mixed-acid indicator has been observed in as little as 1 day. Refrigeration of the mixed-acid indicator has been indicated to help to preserve the stability.

The interferences of aluminum, phosphate, and free chlorine are critical, varying with their concentration and the concentration of fluoride. The percentage of the latter complexed is constant for a given aluminum concentration within the limits of experimental error, providing a procedure for the limited application of corrections for aluminum. The interference caused by phosphates is particularly complex, not lending itself to corrections. Thus, when the phosphate in the sample aliquot exceeds 0.20 mg, the fluoride should be isolated by distillation. The presence of free chlorine is not serious, as it may be removed readily.

Although many methods on the determination of fluoride in water have been published, it is expected that better methods are still to be evolved. The principal problem has been the effect of substances interfering with the fluoride reaction. Among the rela-

TABLE 4

Effect of Hydrogen Ion Concentration

Sample *pH*	Fluoride Error (Positive) *mg/100 ml*
7.0	0.00
6.0	0.00
5.0	0.00
4.5	0.00
4.0	0.02
3.5	0.02
3.0	0.04
2.8	0.06
2.5	0.10
2.2	0.12

tively recent developments has been the zirconium-eriochrome cyanine procedure published by Megregian (*5*) in 1954. Up to the present, considerable study and experience have been necessary with the fluoride procedures to define all of the conditions adequately.

The determination of fluoride with zirconium-alizarin has been studied by a number of investigators. This paper represents a summary and evaluation of 15 years' study and use of the procedure, the results of which should prove very valuable to those utilizing the methods presented in the tenth edition of *Standard Methods.*

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Benefits of Steel Construction in Distribution System Storage

E. H. Aldrich

A paper presented on Feb. 9, 1955, at the New Jersey Section Meeting, Newark, N.J., by E. H. Aldrich, Vice-Pres. & Chief Engr., American Water Works Service Co., Philadelphia, Pa.

THE storing of water was undoubtedly the origin of water systems. When early man made a crude receptacle to collect rain or dip water from a stream, the purpose was basically the same as that which storage serves today: to have a supply available when needed. It was probably many centuries before other elements of a system first came into being. Storage, an essential part of a modern system, is becoming more and more important as population growth, expansion of service areas, and additional uses increase the demand.

The major purpose of distribution storage is to be able to process the water, in advance of actual need, at times when unused facilities are available, as well as to be able to distribute and store water at points in the service area close to the ultimate user. Such storage confers the following principal benefits:

1. A reserve supply is provided in the distribution system for contingencies and for fire-fighting purposes.

2. The demands on the source of supply, production works, and transmission and distribution mains are more nearly equalized. As the facilities are used to better advantage, they need not be so large as otherwise.

3. System flows and pressures are improved and better stabilized to serve the customers over the entire service area.

Little more than a generation ago, in other than gravity systems, the principal motive power was steam, which drove positive-displacement pumps. Their speed could be easily controlled, along with the output and discharge pressures necessary to serve varying needs directly from the pumping station. Many old installations were direct pressure plants without distribution storage. A shutdown could be caused only by a major catastrophe. Since the almost universal adoption of the centrifugal pump, generally driven by electricity, extreme pressure and flow variations in pumping are no longer possible. Moreover, electrical outages are by no means uncommon. Therefore, in small and medium size plants, to meet contingencies and provide adequately and economically for the varying demands, storage has become almost a necessity. When it is properly located and functioning, in adequate amounts, it has the economic value of reducing capital outlay and operating costs of the producing and distributing facilities. These savings are generally far in excess of the extra

cost of installation and maintenance. A fire-fighting standby and a safeguard against interruptions in service are created, greatly contributing (at a low expense) to the reliability and continuity of service.

Storage Related to Demands

For most of the year, service requirements do not change very much from day to day, even through there are substantial variations throughout the day. In most areas of the United States there is a period of 2–3 months when the use of water increases greatly, sometimes exceeding, for shorter periods, twice the average annual rate. During this time, most utilities experience difficulties in meeting the demands.

TABLE 1

Ratio of Period Rates to Average Annual Rates of Production

Period	Maximum Rate % of annual avg
1 year	100
9 months	102
6 months	105
3 months	109
1 month	113
2 weeks	120
1 week	125
5 days	132
1 day	142
18 hr	150
10 hr	170
5 hr	188
2 hr	195
1 hr	200

In planning and designing various elements of the system, experience indicates there are certain percentage relationships between maximum rate to be expected for various periods, compared to the average annual productions (see Table 1). These averages vary little from year to year in a group of about 130 operating systems (representative of the Northeast) supervised by the author's firm. They range from the very small plant to moderate-size systems producing an average of 50 mgd.

Obviously, the averages presented in Table 1 are of little value in planning for a particular system. Each will have its own demands, influenced by size, location, area, type of community, ratios of domestic, commercial, and industrial uses. Of greatest influence on peak use are sprinkling, irrigation, and air-conditioning requirements. Systems with domestic loads in high-value residential developments may have peak hourly demands exceeding five times the average annual use. The systems with a high ratio of industries working around the clock, 7 days a week, may have peak hourly loads as low as 120 per cent of average.

The amount and location of the storage to be provided is therefore tied in very closely to the water consumption throughout the day and in different parts of the system. Rarely can storage be economically justified in much greater amount than will take care of the hourly variations during the day and provide some reserve for fire-fighting purposes and minor contingencies. The storage should normally be replenished each night.

Fire and Normal Service

Storage available for fire fighting is of considerable value in providing the required flows and pressures without burdening the other elements of distribution. This value is recognized by the National Board of Fire Underwriters in rating the fire-fighting ability of a water system (*1*). Storage must provide for fire fighting and

a supply to equalize rates and pressures in normal service. Thus, the superintendent or operator of a system has a problem. His natural tendency is to keep the storage as intact as possible for emergency purposes. The administrators and engineers, however, have furnished the operator a tool whose partial purpose, at least, is to secure capital and operating economies. The latter calls for storage to be used each day to equalize the production output. For most of the year, this use may be minor, and it may even be possible in small plants to eliminate some shift operations, relying entirely upon storage during such periods. In the peak summer periods, however, there is usually a substantial draft from storage to approach an equalization of production, resulting in depletion of the reserve.

From 30 to 40 per cent of the total available storage should, if necessary, be used during the peak periods to help in equalizing production. With adequate storage, a substantial reserve is left for fire-fighting purposes and other contingencies. The attempt to keep a full reservoir, unless it is extremely limited, does not get the most value out of the facilities.

It is not this paper's purpose to set forth in detail how much storage is necessary to produce a constant production rate throughout the day. So much depends upon local conditions, that any such study would have limited application. Experience indicates that in a more or less average system, an effective and usable reserve of 15–20 per cent of the day's production should result in giving a uniform rate. If a maximum 5-day period is taken as the demand base, storage for this purpose requires from 20 to 26 per cent of the average annual rate. If the maximum day's production is the base, the percentage ratios vary between 21.5 and 28.5 per cent.

In a system serving a population of 80,000, the average annual production rate would approximate 10 mgd and the maximum fire demand would be 8,000 gpm for a period of 10 hours. To provide complete coverage for equalizing production rates (in addition to the desired fire flow) during a 5-day maximum-demand period, would require a total usable storage of about 7.3 mil gal, 2.5 mil gal of which would be used for regulation, with 4.8 mil gal used for fire protection. Obviously, if 4.8 mil gal is taken from reserve on a day of maximum demand, pumping at a maximum-production rate of 14.2 mgd (maximum day) for the next 4 days, approximately, would be required to refill the reservoir unless the fire demand took place near or during the week-end slack period.

The foregoing is largely theoretical. Practically, the setup is adequate if usable storage in an amount of 50 per cent of the average annual rate of production is provided and some excess reserve production capacity is made available to bridge the deficiency in storage during the few times in a system's history (if ever) that a major conflagration requiring a maximum fire demand for a duration of 10 hr will occur at a time of other maximum demand.

The survey of operating data made by the staff of AWWA in 1950 noted that the amount of distribution storage differed greatly. Of the 390 cities whose storage data can be related to the production data reported in the survey, the total storage provided represented 1.08 days of the average annual production. Of this, gravity storage accounted for 0.73 days, repumped

storage accounting for 0.35 days. There were 51 cities, about 13 per cent of the total, that indicated they had no distribution storage, including such large cities as Atlanta, Chicago, Houston, Los Angeles, Memphis, and New Orleans, as well as New Haven, Conn., San Jose, Calif., Worcester, Mass., and Elizabeth, N.J. There may be very valid reasons why they have provided none, but many municipalities have reported storage equivalent to several days' production.

Location and Types

The proper location of distribution storage is a matter for intensive study to satisfy each individual situation. If any general rules can be of value, they may be set forth briefly as follows:

1. It should be located near the point of ultimate use.
2. It should preferably be located on the far side of the principal demand (away from the point of production) to provide two-way feed to the major part of the system, thereby economizing on pipelines.
3. It should have incoming and outgoing mains of adequate size to discharge the water effectively when required and for refill from the production works, within an allotted period, when depleted.
4. Its elevation should provide, even upon substantial depletion, adequate pressures to as wide an area as possible.
5. It should take advantage of the topography of the area served by the system, both in elevation and location, to serve effectively as large a section as possible with the most economical type of construction.

The principal types of storage facilities in common use are as follows:

1. Earth embankment reservoirs, generally uncovered.
2. Concrete ground storage reservoirs, covered and uncovered, standard and prestressed reinforced.
3. Steel ground storage reservoirs, gravity and repumped types.
4. Steel elevated tanks and standpipes.
5. Steel pressure storage tanks in small booster areas.

Earth embankment reservoirs. The earth embankment reservoirs were probably the first type of distribution storage in common use. They were generally placed on the higher ground in or near the service area serve the system by gravity, floating on the hydraulic elevation of the system, filling and discharging according to the demand. The early ones were generally constructed in a favorable location by partial excavation and embankment, were almost universally open, and were paved with stone riprap and, later, concrete on the slopes and bottoms.

These installations were, and still are, in most instances, very effective and valuable. Where topography was favorable near the service area, they could be constructed very economically —particularly in the larger sizes. Even so, they are seldom built in present practice. Those still in service are being rapidly retired for a number of reasons, principally because the water is susceptible to pollution and deterioration. Moreover, there is usually a certain amount of leakage, along with an insufficiency of pressure for modern distribution. Also, since the rather common use of sodium hexametaphosphate for specific treatment problems, the increased algal growths have produced deterioration in the storage water.

An AWWA committtee report (*2*) recommends that storage reservoirs be covered. Many state health depart-

ments have prohibited their keeping in service unless covered. The cost of placing concrete roofs over most of the reservoirs is exceedingly high, so it is generally cheaper to replace the capacity, in part at least, with steel storage reservoirs.

Concrete ground storage reservoirs. Probably the second earliest type of container to be commonly used was the concrete ground storage reservoir, at first sometimes uncovered, but later generally roofed over by wood or concrete. Such storage, open, has about the same limitations as the open earth embankment reservoirs, but when covered, the sanitation problem is practically eliminated. The leakage problem is solved by the tanks' fairly impervious bottoms and sides.

These reservoirs, however, were generally limited to a depth of 20–25 ft because of the difficulty of design and need for maintenance of water tightness. They are susceptible to damage by earthquake, and when located in the northern section of the country and exposed to the freezing action, eventually produce a difficult and costly upkeep problem. Within the past few years, circular, prestressed-concrete reservoirs have been successfully erected, with considerable savings over the usual reinforced type, but are not fully competitive in cost with steel construction. Where exposed to severe freezing conditions, reservoirs still have a potential risk of damage, which only time can demonstrate to be imaginary. Where located in warmer climates or where completely covered with earth, prestressed reservoirs and tanks should fill an important need.

Steel storage facilities. Steel reservoirs and tanks have rapidly forged to the front in providing a wide variety of storage holders for use at various points. The more common facilities include steel ground storage reservoirs, elevated tanks, standpipes, and closed steel pressure storage vesssls for remote or unduly elevated sections of the service area. Steel has many advantages in construction, and just one major disadvantage. The modern, welded steel reservoir or tank is permanently watertight, can be made in the various shapes and sizes used in water service, has strength and limits of safety that are well defined and controlled, and at the present time is the most economical type of construction material for this purpose. Unfortunately, the common grades used are subject to corrosion, with maintenance in the form of painting a periodic necessity, although cathodic protection of the interior of the tank assists materially.

Substantially all of the storage facilities which the American Water Works Service Co. has installed in the past 10 years have been of steel construction. Facts on some of these and others throughout the country are presented in Table 2, giving a fairly good survey of present-day cost of various sizes and types. As may be noted, ground storage installation costs just 20–25 per cent of elevated tank erection. Where the situation warrants, the former is the more preferable and economical type.

In providing ground storage where topography does not permit location at or near the hydraulic elevation of the system, booster pumping is required. The relative economy and desirability of pumped as against elevated storage must be determined in each individual case. Usually, the former, even including the capital and operating cost of the booster, will be found to be the more economical. This must be

weighed against the greater dependability of elevated storage. Where substantial reserve is necessary, a combination of the two may be the best solution in many cases.

In determining whether to erect an elevated tank or a standpipe, again the local conditions will decide. Other things being equal, the author prefers a standpipe with the storage above the desired elevation equivalent to what would be provided by an elevated tank.

Most systems, particularly the older ones, have some storage facilities which—because of increased pressures or enlargement of area served—have fallen below the general hydraulic elevation. Many times, only the top few feet of storage may be useful. In a number of companies these installations have been brought back to usefulness by providing automatic booster stations. Proper pumps and controls

TABLE 2

Steel Storage Facilities

Location	Type of Installation	Year Installed	Capacity *mil gal*	Total Cost—$
Mifflin Township, Pa.	Tank	1952	7.26	325,970
Greentree Borough, Pa.	Elevated tank	1953	2.5	363,390
Alexandria, Va.	Elevated tank	1949	1.0	150,800
Ashtabula, Ohio	Clear-water reservoirs & steel purification units	1952	0.48	—
S. Pittsburgh, Pa.	Elevated tank	1953	2.5	323,450
Alton, Ill.	Standpipe	1950	1.54	97,636
Muncie, Ind.	Elevated tank	1954	2.0	306,000
Lexington, Ky.	Two ground storage units	1948	1.0 each	64,200
	Elevated tank	1954	0.5	89,790
Hampton, N.H.	Elevated tank	1953	0.5	92,160
Cairo, Ill.	Elevated tank	1954	0.2	46,710*
Virginia	Ground storage reservoir	1952	2.0	59,345
Wichita, Kan.	Elevated tank	—	2.0	262,776
Westmoreland County, Pa.	Reservoir	1950–51	5.0	130,960
	Reservoir	1950–51	2.5	80,596
	Standpipe	—	0.1	9,225

* On existing foundations.

Frequently bids are called for on the two bases. There are some desirable features of standpipes, compared to elevated tanks, such as having generally two to three times the amount in storage for exceptional demands, perhaps a catastrophic fire, even though the pressures may not be fully adequate. In cost, the break-even point between the two installations occurs between 70 and 80 ft; below, a standpipe is usually cheaper; above, the elevated tank.

make it possible to utilize substantially all of the below-pressure storage. The general scheme is to have the booster start and stop according to the distribution system pressures. An altitude valve on the storage tank is closed by a solenoid when the pump is running. For the refilling operation, the valve does not open until distribution pressures exceed a certain desirable minimum, the control closing when storage is fully replenished. This arrangement

works very well, having been used on a number of outmoded installations.

Steel pressure tanks for booster areas. Customers located on the highest or remotest section of the system desire the same standards of service that others, more favorably located, receive. Most utilities are faced with the problem of providing adequate service to these few customers. Obviously, it is not desirable to increase pressures throughout the system. One answer has been to provide a small underground booster installation involving a steel pneumatic-pressure tank and a pump which operates automatically by pressure control. Properly designed, this is fully effective in normal domestic service. If fire protection is also required, it is furnished by placing a hydrant on each side of the valve segregating the area; the fire department will pump from one hydrant to the other with pressures adequate to supply direct fire streams.

At this time there seems to be no slackening of the demand for improved and additional facilities. The American Water Works Service Co. properties have found it necessary to add substantially to distribution storage in order to render adequate service. Steel reservoirs, elevated tanks, and standpipes have proved completely satisfactory in this program.

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Discussion

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One of the things the writer might enlarge on slightly is the relation of adequate storage and good pressure to automatic sprinklers in small industrial plants, stores, warehouses, and the like. Many of these are more or less borderline cases when it comes to installation of such systems. Unless a municipality has sufficient storage and pressure, sprinklers are not feasible without a secondary source of supply, either an elevated gravity tank or a flat-bottom suction tank and fire pump. In a small business the reduction in insurance rates would not justify the increased expenditure. When increased storage and pressure are provided in the community water system, sprinkler setups are practical without additional cost to the owner. This not only gives an added sense of security, but also often reduces the insurance rates as well as attracting new industry into the area.

Almost all good citizens are anxious to have adequate storage and good pressure, but they often raise serious objections to tank location. Apparently, everyone recognizes the improvements that would come from a strategically placed storage tank, but thinks it should not be placed near his own house. In one instance, a civic group instituted proceedings against the water company and the contractor. The chief cause of complaint was that the tank interfered with television reception.

During the past 25 years, especially, designers and erectors of storage tanks have given more and more thought to

enhancing their beauty, particularly in municipal installations. The factory look is being taken away from elevated tanks and standpipes. The use of welding as a means of construction has allowed the creation of designs that were not considered possible in the days of the riveted tank. Structures are now of all-welded construction, columns are streamlined and tubular, standpipes have ornamental dome or ellipsoidal roofs, and some are ornamented with architectural pilasters, cupolas, and so forth. This does not add greatly to the cost, but goes a long way in establishing better public relations. Since AWWA has issued standard specifications (*1*) for elevated steel tanks and reservoirs, their design has become stabilized.

Some rule of thumb figures for the cost of large steel reservoirs and elevated steel water tanks may be of interest here. On the basis of flat-bottom, all-welded reservoirs, about 40-ft high, completely erected and painted, but not including foundations, piping, or valves, the cost of a 500,000-gal storage vessel would be in the neighborhood of 4–5 cents per gallon; and of a 5-mil gal ground reservoir, about 2½–3 cents per gallon. Elevated steel water storage tanks, 100-ft high to the bottom, cost about as outlined by Aldrich: a 100,000-gal elevated tank would run from 25 to 28 cents per gallon; a 1-mil gal elevated tank, from 15 to 18 cents; and a 2½-mil gal elevated tank, from 10 to 11 cents. On such tanks the entire capacity is usable, of course, with a minimum head of 100 ft.

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George M. Haskew

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It is quite a task to discuss a paper which reflects engineering ability and experience. Distribution storage is of great importance in the operation of all water companies. The writer thoroughly agrees with Aldrich about the kind and proper location of containers. It is certainly becoming very difficult to secure sites, because of the rapid growth and development of the nation, especially metropolitan areas, along with the public's resistance to having construction near its homes.

The writer believes that steel containers are the best to use for either underground or overhead storage, taking into account original cost and maintenance. The writer's firm has an underground storage reservoir of reinforced concrete, with roof of the same material, but all later additions—standpipes, overhead tanks, and other containers—have been constructed of steel, following the trend for better service and economic operation.

A very interesting part of the paper is Table 1 on the ratio of period rates to the average annual rates of production (see p. 574). The use of water is constantly increasing due to lawn watering, air conditioning, and other water-using devices. The figures in Table 1 will not apply exactly to any system, but if they are compared with the data of a particular utility, they will fit fairly well and can be a guide for future operations and additions.

Design of Wye Branches for Steel Pipe

H. S. Swanson, H. J. Chapton, W. J. Wilkinson, C. L. King, and E. D. Nelson

A contribution to the Journal by H. S. Swanson, Structural Eng. Assoc.; H. J. Chapton, Project Design Engr.; W. J. Wilkinson; C. L. King; and E. D. Nelson; all from Los Angeles Dept. of Water & Power, Los Angeles, Calif.

DURING the past 30 years a number of countries have been doing independent work on wye branch design. Work in Europe has been carried on primarily by the Swiss and the English. In 1928 the Swiss firm of Sulzer Brothers began using a circular-collar stiffener on wye branches to be installed in the Puente Nuevo power station in South America (*1*). A patent for this type of reinforcement was granted Sulzer Brothers in 1931 by the Swiss government. Several years later the countries of central Europe began using two- and three-plate reinforcing designs for wye branches, with the Swiss taking the lead in experimental work. The results of some of this work were published (*2*) in 1941.

The English began experimenting with wye branch reinforcement in 1934, without the knowledge that Sulzer Brothers were already acquainted with the problem. This early work of the British was done on small-size pipe; they established a system of basic design for one- and two-plate reinforcement for wye branches. J. S. Blair was mainly responsible for carrying on this work. In 1938 the investigators were granted a British patent. During World War II, work on the experiments halted, and it was not until 1946 that it began again. A much broader program was then instituted, and large-diameter pipes were brought into the investigation. The results of this program were published (*3, 4*) in 1946–47.

Prior to World War I the Southern California Edison Co. had a large-diameter wye branch fabricated at the Krupp Works in Germany. A forge-welded penstock fitting with a cast-steel crotch section, it has been in service in Big Creek Plant No. 1 since 1913. In the following 10 years, several other wye branches were built for this company by M. W. Kellogg Co. The static pressure required of these sections varied from 1,700 to 2,400 ft of head.

Early in 1936 the Metropolitan Water Dist. of Southern California began work on a series of large-diameter wye branches. The first was a 10-ft diameter pipe branching into three 6-ft diameter pipes. The reinforcement was unusual in that it was mostly inside the section and so placed that it did not encroach on the area of flow to any great degree. It was installed in the Hayfield Plant in 1936, and since that time a number of similar sections have been fabricated and put into service.

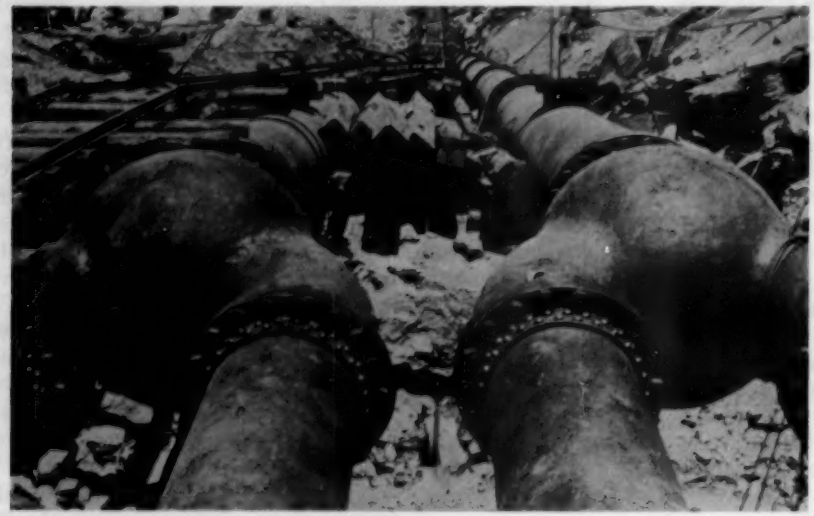
In 1930 the Los Angeles Dept. of Water and Power began investigation and construction of stiffener-reinforced

wye branches. In the following year a system of basic design and stress analysis was devised, and in May 1935 the first major hydraulic test was made. The wye branch, a 36 × 36-in., 45-deg section, was checked for stress distribution by strain gages placed on the reinforcing plates, and the results were analyzed in a report. This unit was later installed in the city's distribution system. Since 1935 this method of using plate stiffeners to reinforce the crotch seams has been used in the distribution system whenever possible. A postwar construction program has resulted in the installation of more than 100 stiffener-reinforced wye branches since 1947.

In the past each wye branch at Los Angeles was designed individually with respect to sizes and conditions, and, as a consequence, there was no opportunity to correlate the data. When the need for building a number of wye branches at the same time arose, it was decided to consolidate the design work and testing analysis into a report which covered a range of sizes and pressures.

General Purpose

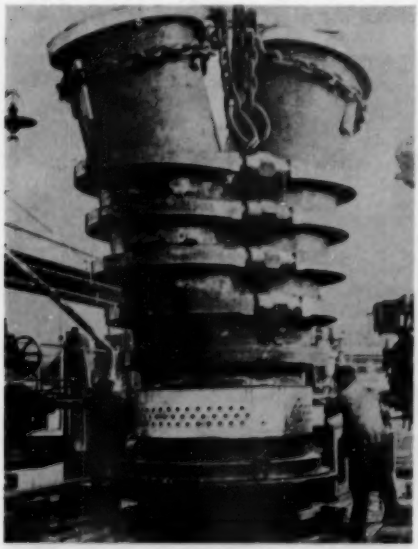
The general purpose of this paper is to present a practical and economical method to be used in designing large-diameter wye branches. In attempting this, the authors have sought to establish standards and good practice in determining the proper type of reinforcement and other basic facts connected with the design. Tables, charts, and nomographs have been included to simplify the work as much as possible.



1913 Installation

The above sections were fabricated for the Southern California Edison Co. by the Krupp Works of Germany prior to World War I. They were installed in Big Creek Plant No. 1 in 1913 and have been in continuous service ever since. The breech joint is cast steel, and the penstock pipe is forge welded.

The general assumptions in this report are based on the use of a tested model that is similar to the actual wye branch to be designed (*5*). The actual pipe and the model must be fabricated from the same material so that the modulus of elasticity, modulus of rigidity, and Poisson's ratio may be given consideration in the prediction equations. The free ends of the unequal-length stiffeners are assumed to be affected by a balancing force which causes equal deflections. This assumption is made so that the principles of statics can be applied to the problem. The main concern is finding a means of transmitting circumferential stresses due to hydrostatic load into the stiffener beams. Temperature is ignored in the design because the effect is negligible when the section is underground. If the section is to be laid aboveground, however, temperature must be included in the design or allowed for through the use of expansion joints.



1928 Installation

This band-reinforced wye was also fabricated for the Southern California Edison Co. by Krupp. It was placed in operation in Big Creek Plant No. 2A in 1928. The large penstock is 3 in. thick and the two 48-in. branches are 2¼-in. thick.

1936 Installation

This juncture of three 6-ft diameter pipes entering a 10-ft diameter line was fabricated in 1936 for the Hayfield Pumping Plant on an aqueduct of the Metropolitan Water Dist. of Southern California. The vertical wye stiffener plates continue through the pipe as tension diaphragms and are tied together on the uphill end by the circumferential stiffener ring.

Design and Test Methods

The design method developed for the two-plate wye section involves estimating the amount of plate needed for the reinforcement and applying the resultant circumferential stress from the pipe shells on it. In a tee section, the curved-beam equations could then be used to calculate the critical stresses.

The 24 × 24-in., 45-deg wye branch at left shows the two stiffener plates in place. The inside curve of the plates match the inner surface of the pipe shell.

At right, the branch pipe is being lowered into position.

At left, the assembled unit is being fitted for welding. If a third plate were to be added, it could be welded on when this portion is completed.

Fabrication Methods

When the section is other than 90 deg, however, one plate will have a larger load than the other, so that the calculations must include balancing the deflections of both plates at the junction in order to arrive at the amount of load which the longer plate will transfer to the shorter plate. In dealing with the larger-diameter pipe, many designs which would hold the deflections down were tried. The most economical of these proved to be the three-plate design in which a half-ring plate was added to a two-plate design to hold down the free ends.

The design of the single-plate wye began with the supposition that the plate could be treated as a ring girder (*6*). Upon examination of the problem, it was concluded that the plate could vary anywhere from a ring girder to a two-plate design, depending upon the ratio of pipe diameters. Equations were set up under this assumption and some designs tested. As a result of this work, a graph was made which is used in conjunction with the nomograph.

1935 Installation

In 1935 this section was tested hydrostatically for stress distribution by the Los Angeles Dept. of Water and Power. Following the test, the stresses were analyzed in a report and the section was put into service in the distribution system.

As an aid in devising a practical method of design, several examples are given, with accompanying tables and sketches. The nomograph section of the report contains simplifications which give results within the range of accuracy necessary, but, to use the nomograph (Fig. 15) properly, a few

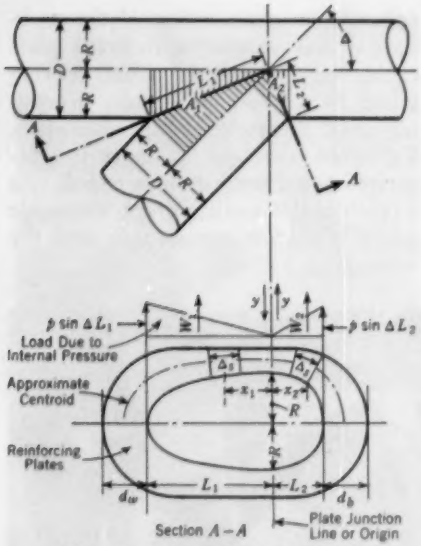


Fig. 1. Intersection of Equal-Diameter Plates

Δs is unit length along centroidal axis; x is horizontal distance to center of Δs.

facts must be noted. First, the depth of plate read from the nomograph is for 1-in. thick plate; when this is used with pipe of a very large diameter, the curved-beam K factor (*7*) becomes very great, increasing the critical stress. If the value from the nomograph is converted to a thicker plate with a smaller depth, however, the K factor is lessened, giving a better design. It is recommended that the depth of plate, d_w, should not exceed $30t$ (t equals thickness of crotch plate).

Second, under some very high head conditions in smaller-diameter pipe, the depth of plate has a tendency to approach the diameter of the pipe. This also will increase the K factor. It is recommended, in such a case, that the depth of plate be made less than the diameter of the pipe.

If these recommendations are followed, the nomographic design will usually include a safety factor which will keep stresses below the yield point of structural steel. Because the maximum allowable stress of the steel used in this report is 30,000 psi, however, it should be understood that the design pressure used in the nomograph was kept to 1.5 times the working pressure

TABLE 1

Explanation of Symbols

Symbol	Explanation	Units
Δ	deflection angle of pipeline	deg
D	diameter of pipe	in.
R_B	radius of large pipe	in.
R_s	radius of small pipe	in.
R	centroidal radius of reinforcing plate	in.
a	inside radius of reinforcing plate	in.
d	depth of reinforcing plate	in.
d_1	d at 1-in. thick plate (as in nomograph)	in.
d_w	d at acute crotch	in.
d_b	d at obtuse crotch	in.
d_t	d at junction of reinforcing plates	in.
L_1	length of axis of large reinforcing plate	in.
L_2	length of axis of small reinforcing plate	in.
W_1	total water load on large reinforcing plate	lb
W_2	total water load on small reinforcing plate	lb
y_1	theoretical reaction on large reinforcing plate	lb
y_2	theoretical reaction on small reinforcing plate	lb
y_3	theoretical reaction on ring plate	lb
$T=P$	resultant load on horizontal axis of plates (equal to $W \pm y$)	lb
p	internal pressure in pipe	psi
t	thickness of reinforcing plate	in.
A	cross-sectional area	sq in.
M	moment in section due to load	in.-lb
m	moment at any point due to load of unity where deflection is to be found	in.-lb
E	modulus of elasticity for steel	psi
u	Poisson's ratio	

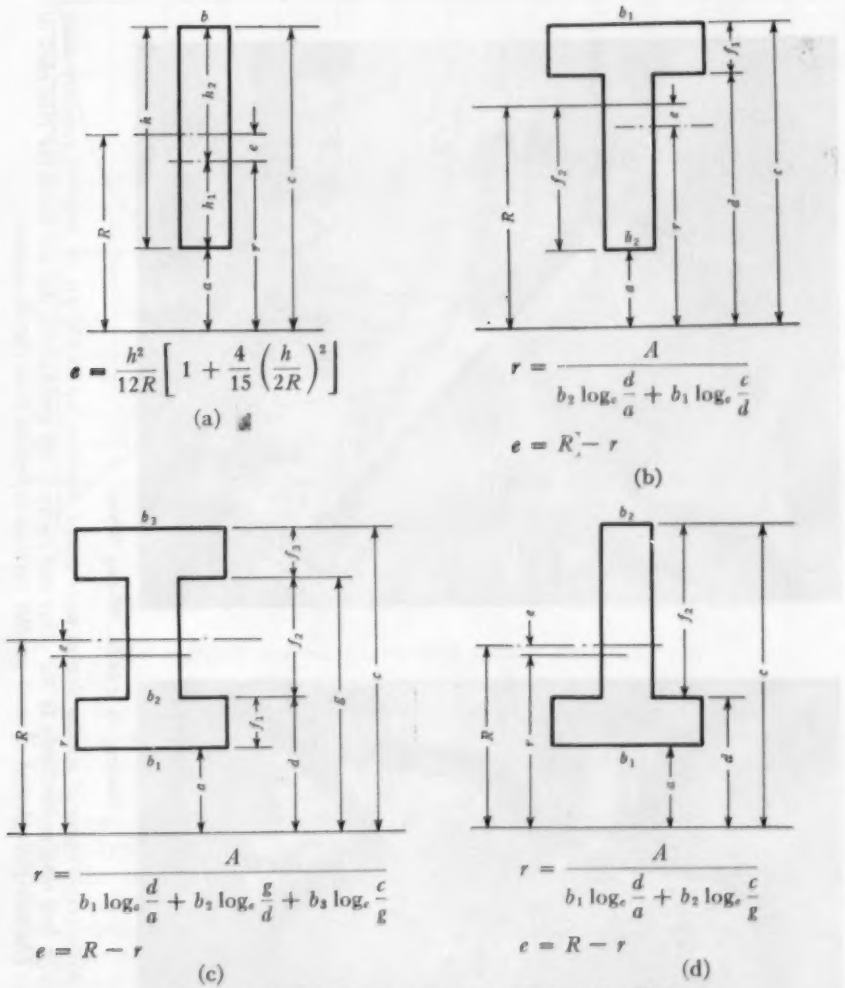

$e = \frac{h^2}{12R}\left[1 + \frac{4}{15}\left(\frac{h}{2R}\right)^2\right]$
(a)
$r = \frac{A}{b_2 \log_e \frac{d}{a} + b_1 \log_e \frac{c}{d}}$
$e = R - r$
(b)
$r = \frac{A}{b_1 \log_e \frac{d}{a} + b_2 \log_e \frac{g}{d} + b_3 \log_e \frac{c}{g}}$
$e = R - r$
(c)
$r = \frac{A}{b_1 \log_e \frac{d}{a} + b_2 \log_e \frac{c}{g}}$
$e = R - r$
(d)

Fig. 2. Typical Cross Sections of Reinforcing Plates

in order to approximate an allowable stress of 20,000 psi. Another factor to be noted is that, although the designs used in the report are based on a rectangular cross section for the crotch plates, the tests showed that the welding in the crotch, plus a portion of the pipe shell, acted as a flange which decreased the inside fiber stress. This was taken into account through the use of a factor based on the data.

The various designs were tested by placing the section under hydrostatic load and using electrical strain gages to indicate the amount of stress to which the section was subject (*8*). These data were compared with the design stresses, and the method of design was corrected to conform to the results of the tests.

Testing 24 × 24-in., 90-deg Wyes

A two-plate (left) and a three-plate (right) wye of similar design were tested together, up to 550 psi, in order to compare their stress patterns. The third plate, a half ring welded only to the other two plates at top and bottom, did not touch the pipe shell at any point. The two-plate design showed greater vertical deflection than the three-plate.

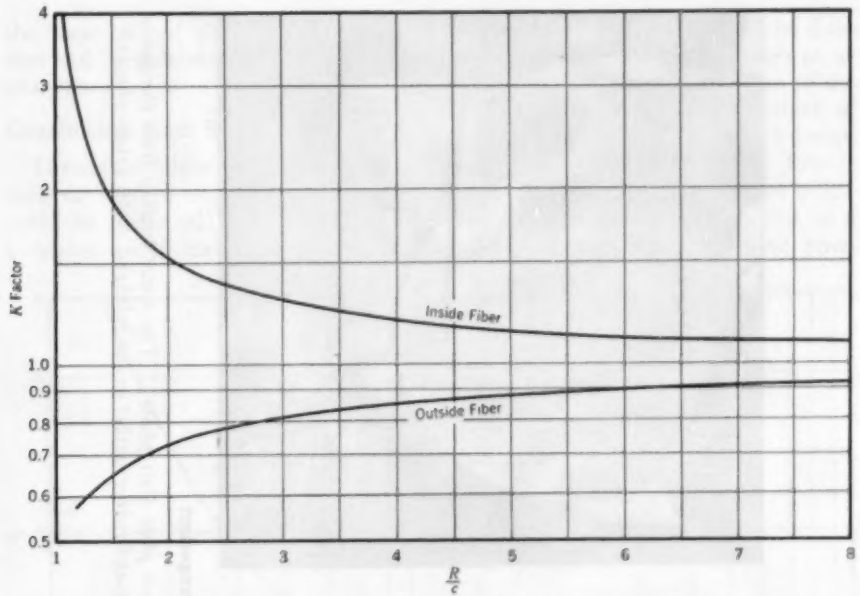


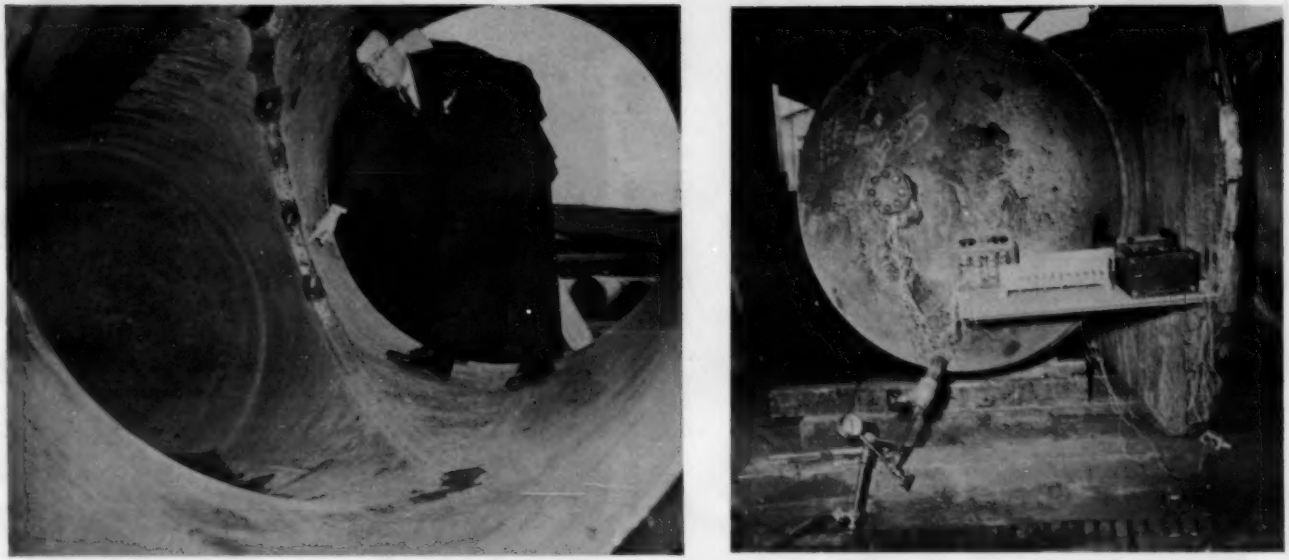
Fig. 3. *K* Factor for Curved Beams (Rectangular Cross Section)

R *is radius of centroid;* c *is one-half depth of plate;* $f_{max} = K \frac{Mc}{I}$.

The use of single-direction SR-4 strain gages in the testing of the wye sections was desirable because this type of gage required less mounting room and fewer connecting wires than the rosette type and gave stress results while the section was still under test. The Baldwin-Lima-Hamilton Corp., Philadelphia, developed a method of using these gages on sections with bi-directional stress, but the direction of the principal stress must be known. The rosette gages are essential for obtaining stresses in locations where the direction of the principal stress is not known. The equations used in computing stresses from rosette strain readings are so involved that the calculations had to be made in the office after the tests had been run (*8, 9*).

During the tests a waterproof cement was found which allowed the SR-4 electrical strain gages to be attached to the inside periphery of the crotch stiffeners without shorting out under the high hydrostatic pressures used. Single SR-4 gages applied in this manner gave test results in sections that had not previously been available. (See photos on pages 590 and 594.)

On one large wye branch, the Magnaflux Corp., Chicago, was called in to use the "Stresscoat" method for determining the stresses, in conjunction with the electrical strain gages which were normally used. It was considered advisable to try this method because of the complete coverage that it gives. The results of the two methods compared very favorably and gave a good picture of the stresses in the section. The "stresscoat" test confirmed



Strain Gage Test Arrangement

The electrical strain gages were placed on the inner curve of the crotch plate (left photo). The wires leading from the inside gages out to the meter were compressed between two layers of a cork-and-rubber gasket in the bulkhead outlet (right photo).

the locations of the critical stresses that had been determined in the design analysis.

Conclusions and Recommendations

Throughout these tests, it was found that the design of the actual section could be predicted from the model to a degree within the range of accuracy required. Because of the difficulties encountered in analyzing the type of curved beam used in these designs, however, it became necessary to apply stipulations to the design procedure in order to conform to the test results.

It was found relatively easy to obtain good balance in the two-plate design, but, in the three-plate design, it was apparent that there would be difficulty in proportioning the plates so as to have each one carry its share of the load. By the use of the method of balancing deflections, a balanced design for a three-plate section was found; because the reduction in the size of the plate depths, compared with that of a two-plate design, was very small, however, it was concluded that, if a third plate was to be used to hold down deflections on larger-diameter pipe, it should be added to the two-plate design and the dimensions should conform to the top depth. Also, it should be welded to the other reinforcement plates only at top and bottom, being left free from the pipe shell, so that

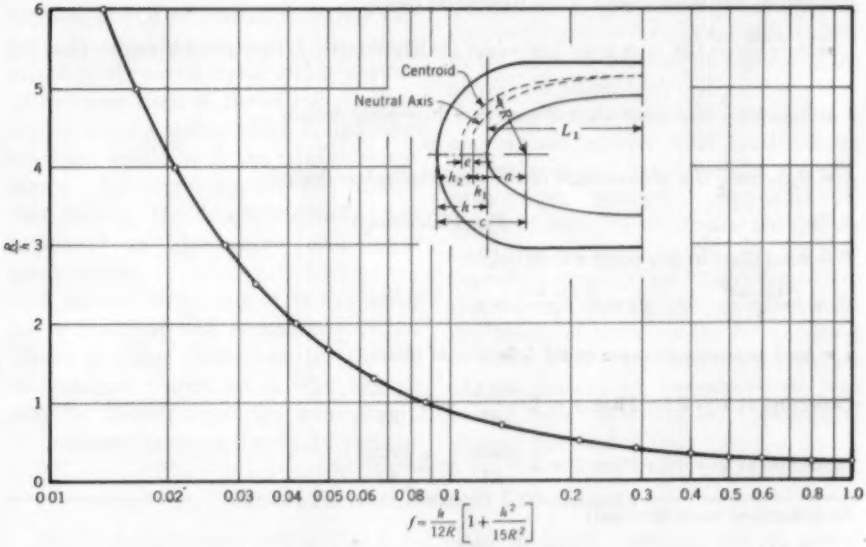


Fig. 4. Determination of Eccentricity, *e*

Based on Timoshenko (10) equation:

$$e = \frac{h^2}{12R}\left[1 + \frac{4}{15}\left(\frac{h}{2R}\right)^2\right]; \quad e = fh; \quad f_1 = \frac{Mh_1}{Aea} + \frac{P}{A}; \quad f_2 = \frac{Mh_2}{Aec} - \frac{P}{A}$$

in which e *is eccentricity (distance from centroidal axis to neutral axis in curved beam);* h *is depth of curved beam; and* R *is radius of controidal axis of curved beam.*

TABLE 2

General Formulas (See Fig. 1)

$$L_1 = \frac{R}{\sin\frac{\Delta}{2}};\quad L_2 = \frac{R}{\cos\frac{\Delta}{2}};\quad W_1 = \frac{p \sin \Delta L_1^2}{2};\quad W_2 = \frac{p \sin \Delta L_2^2}{2}$$

W at any point x from origin (triangular load):

$$W_x = \frac{p \sin \Delta x^2}{2}$$

Moment at any point x from origin (triangular load):

$$m = \frac{p \sin \Delta x^3}{6};\ M = m \pm xy$$ (use minus for larger plate, L_1; use plus for smaller plate L_2)

$$a = R_{pipe} \sin\frac{\Delta}{2}$$ (for acute angle of two- and three-plate design)

$$a = R_{pipe} \cos\frac{\Delta}{2}$$ (for obtuse angle of two- and three-plate design)

Two-Plate Design

P = total shear at any point x from origin

$$P_1 = \frac{p \sin \Delta x^2}{2} - y;\quad P_2 = \frac{p \sin \Delta x^2}{2} + y$$

y = load necessary to cause equal deflection of plates

Deflection at $x = 0$ for Plate 1 $= \Sigma \frac{m_1 x \Delta s}{EI} - \Sigma \frac{y x^2 \Delta s}{EI}$

Deflection at $x = 0$ for Plate 2 $= \Sigma \frac{m_2 x \Delta s}{EI} + \Sigma \frac{y x^2 \Delta s}{EI}$

As deflections must be equal:

$$\Sigma \frac{m_1 x \Delta s}{EI} - \Sigma \frac{y x^2 \Delta s}{EI} = \Sigma \frac{m_2 x \Delta s}{EI} + \Sigma \frac{y x^2 \Delta s}{EI}$$

Three-Plate Design

$y_3 = y_1 + y_2$; Plate 1, $\delta = \Sigma \frac{m_1 x \Delta s}{EI} - \Sigma \frac{y_1 x^2 \Delta s}{EI}$;

Plate 2, $\delta = \Sigma \frac{m_2 x \Delta s}{EI} - \Sigma \frac{y_2 x^2 \Delta s}{EI}$; Plate 3, $\delta = \int \frac{y_2 x^2 ds}{EI}$ (half ring)

Maximum Stresses

$$M_{max} = W\left[\frac{L}{3} + (0.2)\frac{d}{2}\right] \pm y\left[L + (0.2)\frac{d}{2}\right]$$

$$f_{max} = \frac{M_{max} h_1}{Aea} + \frac{W \pm y}{A};\quad f_{min} = \frac{M_{max} h_2}{Aec} - \frac{W \pm y}{A}\ *$$

$$f_{max} = K\frac{M_{max} c}{I} + \frac{W \pm y}{A};\quad f_{min} = \frac{M_{max} c}{I} - \frac{W \pm y}{A}\ \dagger$$

* Timoshenko (*10*); c equals one-half depth of plate; for h_1 and h_2, see Fig. 4 (page 591).
† Seely (*7*).

none of the shell stress would be transferred to the ring plate. It appears practical to begin adding the ring plate to a two-plate design on high-head, equal-diameter wye branches with pipe diameters greater than 60 in.

During early testing of the models, it was found that, in the critical sections, the measured stresses were lower than the design stresses without exception, giving an additional safety factor. It was concluded that the flange effect of the weld metal and a portion of the pipe shell in the crotch had accepted some stresses which it had been assumed would be in the reinforcement plates. By correcting for this in further testing, the design methods were improved to give more economical proportions.

A review of the test data evaluated for a practical and economical design shows that the dimensions for the reinforcement plates for a wye branch may be chosen from the nomograph. If similarity between the model and the actual pipe is maintained, the safety of the structure is assured.

The principal symbols used in the design equations and sketches are listed in Table 1; other symbols will be explained when used. The general formulas employed are shown in Table 2. Following the section on analysis and design procedures is a description of the nomograph method, including a discussion of the development of the curves (pages 617–623). Test results, in the form of drawings and tables, are shown on pages 624–627.

Appendix A (page 628) contains a comparison between a Sulzer Brothers design and one based on the authors' nomograph method. Appendix B (pages 628–629) illustrates the deflections that occurred on two 45-deg wye branch pipes. The deflections are plotted on a nomograph, to provide an indication of what might be expected in other 45-deg sections. A discussion of the results of overstress in wye branch pipes is found in Appendix C (pages 628 and 630).

Analysis and Design Procedures

In the designs described in this portion of the paper, a trial depth and thickness of stiffener plate were selected and the stresses in the critical sections were computed. Then a new depth and thickness were chosen, as indicated by the resulting stresses. These stresses were evaluated by using the loading assumptions shown in Fig. 1, along with a carryover load, y, found by a method of balanced deflections. The solution equations were taken from Timoshenko (*10*) and Seely (*7*).

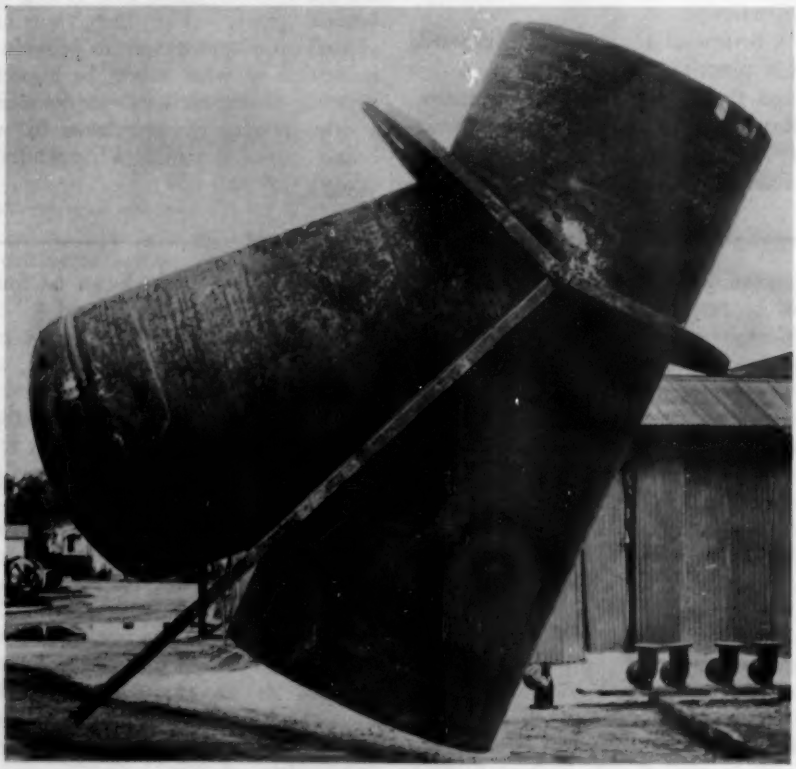
The various cross sections of curved beams which could apply to wye branch design are shown in Fig. 2, although only the rectangular section will be considered in this report. The eccentricity, e, and the K factor which is used in Seely's method can be found from the curves in Fig. 3 and 4.

A series of typical designs are presented: single plate, $\Delta = 90$ deg; two plate, $\Delta = 90$ deg; two plate, $\Delta = 53$ deg; two plate, $\Delta = 45$ deg; three plate, $\Delta = 90$ deg; three plate, $\Delta = 45$ deg; and three plate (alternate design), $\Delta = 45$ deg.

In an early effort to use the model theory as an aid to future design, a number of tables were computed, such as the one in Fig. 6 (page 597). This table was based on the two-plate 24-in. wye branch shown in Fig. 7 and was retained as an example only. The nomograph (Fig. 15) has greater versatility and lends itself better to correlation with test data.

Two-Plate Wye Under Test

This photograph of a 30 × 30-in., 90-deg wye under test shows the mercury switches and deflection gages.



Three-Plate Wye Branch

One-Plate Design

For one-plate design:

$$0.3 \leqq \frac{R_s}{R_B} \leqq 0.6$$

$$W_1 = st_p = pR_B$$

$$W_2 = \frac{W_1}{2} = \frac{pR_B}{2}$$

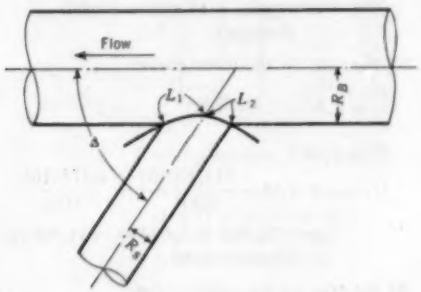
At crown:

$$\text{Moment due to } W_2 = \frac{W_2 R_s^2}{4} \text{ *}$$

$$\text{Moment due to } W_1 = \frac{-W_1 R_s^2}{4 \sin^2 \Delta} \text{ *}$$

$$M_1 = \left(W_2 - \frac{W_1}{\sin^2 \Delta}\right)\frac{R_s^2}{4}$$

$$r_3 = \frac{L_3^2}{R_s} + \frac{d_t}{2}$$



At sides:

$$\text{Moment due to } W_1 = \frac{W_1 R_s^2}{4 \sin^2 \Delta}$$

$$\text{Moment due to } W_2 = \frac{-W_2 R_s^2}{4}$$

$$M_2 = \frac{\left(\dfrac{W_1}{\sin^2 \Delta} - W_2\right) R_s^2}{4}$$

$$r_1 = \frac{R_s^2}{L_1} + \frac{d_w}{2}$$

$$r_2 = \frac{R_s^2}{L_2} + \frac{d_b}{2}$$

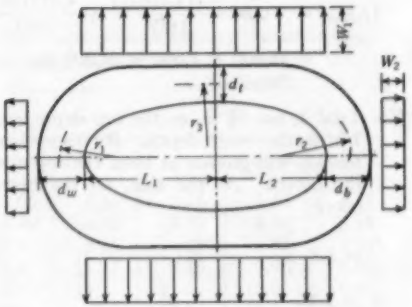


Fig. 5. General Equations for One-Plate Wye Design

Example

Main pipe radius, $R_B = 30.5$ in.
Branch pipe radius, $R_s = 10$ in.
Deflection angle, $\Delta = 90$ deg
Design pressure, $p = 300$ psi

Uniform load at top, $W_1 = pR_B = 300(30.5)$
$= 9{,}150$ psi

Uniform load at side, $W_2 = \dfrac{pR_B}{2} = \dfrac{300(30.5)}{2}$
$= 4{,}575$ psi

Maximum moment at top, $M_1 = (W_2 - W_1)\dfrac{R_s}{4}$
$= -114{,}400$ in.-lb

Maximum moment at side, $M_2 = (W_1 - W_2)\dfrac{R_s}{4}$
$= 114{,}400$ in.-lb

As a trial, use a 6 × 1-in. collar to stiffen the joint. Then:

$$f_{max} = K\dagger \frac{Mc}{I} + \frac{T}{A}$$

$$= K\dagger \frac{6M}{bd^2} + \frac{WR_s}{bd \sin \Delta}$$

At the side, on the inner curve:

$$\frac{R}{c} = 4.2$$

* Paris (*11*).

† K is Seely's constant, dependent on the curvature of the collar.

$K = 1.185$

$$(f_3)_{max} = 1.185\,\frac{114{,}400(6)}{1(6)^2} + \frac{9{,}150(10)}{1(6)}$$

$$= 22{,}600 + 15{,}200 = 37{,}800 \text{ psi (tension)}$$

At the top, on the inner curve:

$$\frac{R}{c} = 4.3$$

$K = 1.18$

$$(f_1)_{max} = 1.18\,\frac{-114{,}400(6)}{1(6)^2} + \frac{4{,}575(10)}{1(6)}$$

$$= -22{,}500 + 7{,}600 = -14{,}900 \text{ psi (compression)}$$

At the top, on the outer curve:

$$\frac{R}{c} = 4.3$$

$K = -0.86$

$$(f_1)_{max} = -0.86\,\frac{-114{,}400(6)}{1(6)^2} + \frac{4{,}575(10)}{1(6)}$$

$$= 16{,}400 + 7{,}600 = 24{,}000 \text{ psi (tension)}$$

For Trial 2, use 5½ in. as the top depth and 7 in. as the crotch depth. By inspection, tension will govern at both the top and the crotch. At the side, on the inner curve:

$$\frac{R}{c} = 3.8$$

$K = 1.21$

$$(f_3)_{max} = 1.21\,\frac{114{,}400(6)}{1(7)^2} + \frac{9{,}150(10)}{1(7)}$$

$$= 17{,}000 + 13{,}000 = 30{,}000 \text{ psi (tension)}$$

At the top, on the outer curve:

$$\frac{R}{c} = 4.63$$

$K = -0.87$

$$(f_1)_{max} = (-0.87)\,\frac{-114{,}400(6)}{1(5.5)^2} + \frac{4{,}575(10)}{1(5.5)}$$

$$= 19{,}700 + 8{,}300 = 28{,}000 \text{ psi (tension)}$$

In the first trial, the outer stress at the side was ignored because it could not exceed the inner stress. At the top, however, the direct stress is additive to the outer stress due to moment, increasing the total stress to the point where it must be investigated along with the inner stress.

The final design is a 1-in. thick collar, welded at the junction of the pipes, with a depth of 7 in. in the crotches and a depth of 5½ in. at the top and bottom.

NOTE: Bier (*6*) states that the assumption of a uniform load is on the side of safety, but it should be noted that, at smaller deflection angles, the ring becomes egg shaped and the ring girder equation for moment becomes less applicable. Also, when the branch pipe diameter approaches the diameter of the main pipe, the assumption that the collar lies in one plane causes inaccurate results. The conclusion is that this method of one-plate design can apply only to limited cases of wye branches where the deflection angle is close to 90 deg and the diameter of the branch pipe is substantially smaller than that of the main pipe.

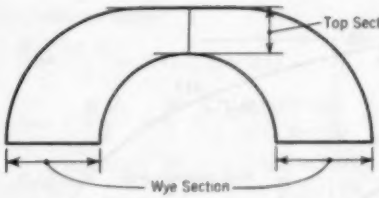
The ASME Boiler Code (*12*) covers a good portion of the one-plate design in the lower ratios because the reinforcement pad is close to the plane of stress. It appears that this method is good up to a ratio of diameters of pipe of 0.3:1. By the same token, the ring girder assumption appears to be satisfactory up to a ratio of 0.7:1, but above this ratio, the curvature of the plate is such that it has been found advisable to use an equal pipe diameter design and a truncated cone to reduce the diameter of the outlet pipe to the desired size. In the one-plate design curve in the nomograph section, an attempt was made to join the methods of design together so that they merge into the two-plate design.

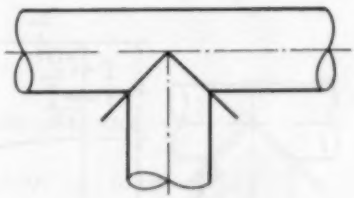
Two-Plate Design

Example 1

$R_B = 12$ in.
$R_s = 12$ in.
$\Delta = 90$ deg
$p = 350$ psi
Pipe shell thickness, ¾ in.
Depth at crotch, 12 in.
Depth at crown, 5 in.

NOTE: As the two plates are identical, it is not necessary to set up a table for the computation of force y, which automatically reduces to zero. The selected depths for crotch and crown were obtained from the prediction tables for pipe tees which ranged in diameter from 12 in. to 120 in., at a pressure of 350 psi.





$S = \frac{bd^2}{6}$ (section modulus)

$S = N^3 S_m$*

$N = \frac{D}{D_m}$

$d^2 = \frac{6S}{b} = \frac{6N^3 S_m}{b} = \frac{6D^3 S_m}{bD_m^3}$

$d = \sqrt{\frac{6S_m}{bD_m^3}}\,(D^3)$

$d = KD^3$

Data:

S_m = 36 in., 6.25

D_m = 24 in.

b = 1, 1½, 2, 3 in.

p = 350 psi

For top section, K equals:

0.0521 at 6.25, 1 in.
0.0425 at 6.25, 1½ in.
0.0368 at 6.25, 2 in.
0.0301 at 6.25, 3 in.

For wye section, K equals:

0.125 at 36, 1 in.
0.102 at 36, 1½ in.
0.0884 at 36, 2 in.
0.0722 at 36, 3 in.

* Subscript "m" refers to model.

Diam. D in.	D^3	Plate Depth, d—in.							
		Wye Side				Top			
		b = 1 in.	b = 1½ in.	b = 2 in.	b = 3 in.	b = 1 in.	b = 1½ in.	b = 2 in.	b = 3 in.
12	41.5	5.19	4.23	3.67	3.00	2.16	1.76	1.53	1.25
24	117.6	14.7	12.0	10.4	8.49	6.12	5.00	4.33	3.54
30	164.4	20.5	16.8	14.5	11.9	8.56	6.99	6.05	4.94
36	216.0	27.0	22.0	19.1	15.6	11.25	9.19	7.96	6.50
42	272.4	34.1	27.8	24.1	19.7	14.2	11.6	10.0	8.19
48	332.6		33.9	29.4	24.0		14.1	12.2	10.0
54	396.9		40.5	35.1	28.7		16.9	14.6	11.9
60	465.0		47.4	41.1	33.6		19.8	17.1	14.0
66	536.0			47.4	38.7			19.7	16.1
72	611.0			54.0	44.1			22.5	18.4
84	770.0			68.1	55.6			28.4	23.2
96	941.0				67.9				28.3
108	1,123.0				81.1				33.8
120	1,313.0				94.8				39.5

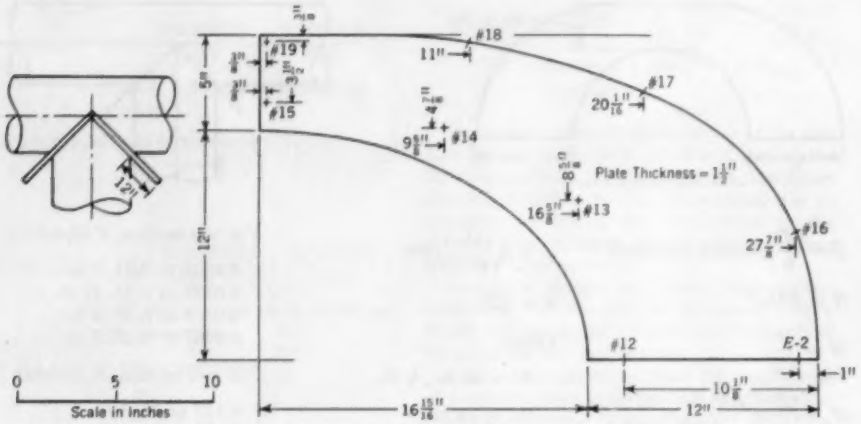
Fig. 6. Two-Plate Design (Example 1)—Plate Depth Calculations

Computation for Selection of Plate Thickness

$$W_1 = W_2 = \left(\frac{p}{2}\right)\frac{(L)^2 \sin \Delta}{2} = \frac{350}{2}(16.97)^2 = 50{,}400 \text{ lb}$$

Maximum moment:

$$M = W_1\left(\frac{L}{3} + (0.2)\frac{d}{2}\right) = 50{,}400\left[\frac{16.97}{3} + (0.2)\frac{12}{2}\right] = 345{,}500 \text{ in.-lb}$$



Item	Stress*—lb		Item	Stress*—lb	
	Computed	Measured		Computed	Measured
#12	9,760	9,260	#17	−8,180	−5,710
#13	7,980	9,250	#18	−5,525	−3,090
#14	3,760	6,490	#19		−3,550
#15		1,760	*E*-2	−4,020	−5,650
#16	−4,880	−4,000			

* At 350-psi pressure.

Fig. 7. Two-Plate Design (Example 1)—Plan of Stiffener Plates

For any pressure other than the design value (350 psi in this instance), after the critical dimensions have been selected from the nomograph (Fig. 15), the outside curve of this plate should conform geometrically with the shape shown.

$$a = R \sin \frac{\Delta}{2} = 12 \sin \frac{90}{2} = 12(0.707) = 8.48 \text{ in.}$$

$$\frac{R}{h} = \frac{14.48}{12} = 1.207; \; e = 0.071(12) = 0.852 \text{ in.}; \; h_1 = 5.148 \text{ in.}; \; h_2 = 6.852 \text{ in.}$$

By Timoshenko equation:

$$f_{max} = \frac{Mh_1}{Aea} + \frac{W}{A} = \frac{345{,}500(5.148)}{12(0.852)(8.48)} + \frac{50{,}400}{12} = 24{,}700 \text{ psi (tension) for 1-in. plate}$$

Use 1½-in. plate:

$$f_{max} = \tfrac{2}{3}(24{,}700) = 16{,}500 \text{ psi}$$

$$f_{min} = \frac{-Mh_2}{Aec} + \frac{W}{A} = -\frac{345{,}500(6.852)}{12(1.5)(0.852)(20.48)} + \frac{50{,}400}{(12)(1.5)} = -4{,}750 \text{ psi (compression)}$$

By Seely equation:

$$\frac{R}{c} = \frac{14.48}{6} = 2.413;\ K = 0.77$$

$$f_{max} = \frac{KMc}{I} + \frac{W}{A} = \frac{1.40(345{,}500)}{0.167(1.5)(12)^3} + \frac{50{,}400}{12(1.5)} = 16{,}200 \text{ psi (tension)}$$

$$f_{min} = \frac{KMc}{I} + \frac{W}{A} = \frac{(0.77)(-345{,}500)}{0.167(1.5)(12)^3} + 2{,}800 = -4{,}600 \text{ psi (compression)}$$

Example 2

$R_B = 18$ in.; $R_s = 18$ in.; $\Delta = 45$ deg; $p = 385$ psi

Pipe shell thickness, $\frac{3}{8}$ in.; depth at crown, 12 in.

Table 3 shows the corrected values as constructed; it also provides a simple expedient for converting the design value of y to a new y for any other pressure desired.

Calculated Stresses for Larger Plate

Depth at crotch, 30 in.; plate thickness, $2\frac{1}{8}$ in.

NOTE: Owing to the flange effect of the pipe shell, a factor of 0.2 is applied to $\frac{d}{2}$. The centroidal axis is taken as being parallel to the curve where the stresses are being computed.

Maximum moment:

$$M = W_1\left[\frac{L_1}{3} + (0.2)\frac{d_1}{2}\right] - y\left[\frac{L_1}{3} + (0.2)\frac{d_1}{2}\right]$$

$$= 277{,}000\left[16.13 + (0.2)\frac{30}{2}\right] - 45{,}800\left[48.38 + (0.2)\frac{30}{2}\right] = 2{,}946{,}000 \text{ in.-lb}$$

$$a = R\sin\frac{\Delta}{2} = 18\sin 22\tfrac{1}{2}^\circ = 18(0.3827) = 6.9 \text{ in.}$$

$$\frac{R}{h} = \frac{21.9}{30} = 0.73;\ e = 0.128(30) = 3.84 \text{ in.};\ h_1 = 11.15 \text{ in.}$$

Maximum fiber stress (tension):

Timoshenko:

$$f_{max} = \frac{Mh_1}{Aea} + \frac{T}{A} = \frac{2{,}946{,}000(11.15)}{30(2.125)(3.84)(6.9)} + \frac{231{,}000}{30(2.125)} = 19{,}950 + 3{,}600 = 23{,}600 \text{ psi}$$

Seely:

$$\frac{R}{c} = \frac{21.9}{15} = 1.46;\ K = 2.01$$

$$f_{max} = \frac{2.01(2{,}946{,}000)(6)}{2.125(900)} + 3{,}600 = 18{,}600 + 3{,}600 = 22{,}200 \text{ psi}$$

Calculated Stresses for Smaller Plate

Depth at crotch, 20 in.; plate thickness, $2\frac{1}{8}$ in.

Maximum moment:

$$M = W_2\left[\frac{L_2}{3} + (0.2)\frac{d_2}{2}\right] + y\left[\frac{L_2}{3} + (0.2)\frac{d_2}{2}\right]$$

$$= 52{,}500\left[7.02 + (0.2)\frac{20}{2}\right] + 45{,}800\left[21.06 + (0.2)\frac{20}{2}\right] = 1{,}530{,}000 \text{ in.-lb}$$

TABLE 3

Two-Plate Design Calculations (Example 2)

$$L_1 = \frac{R}{\sin\frac{\Delta}{2}} = \frac{18}{0.3827} = 47.04; \quad L_2 = \frac{R}{\cos\frac{\Delta}{2}} = \frac{18}{0.9239} = 19.48$$

L_1 (scaled from Plate 1) = 48.38; L_2 (scaled from Plate 2) = 21.06

$d_1 = 30$ in.; $d_2 = 20$ in.

$$W_1 = \frac{p \sin \Delta (L_1)^2}{2} = \frac{385(0.707)(48.38)^2}{2} = 318{,}500 \text{ lb}$$

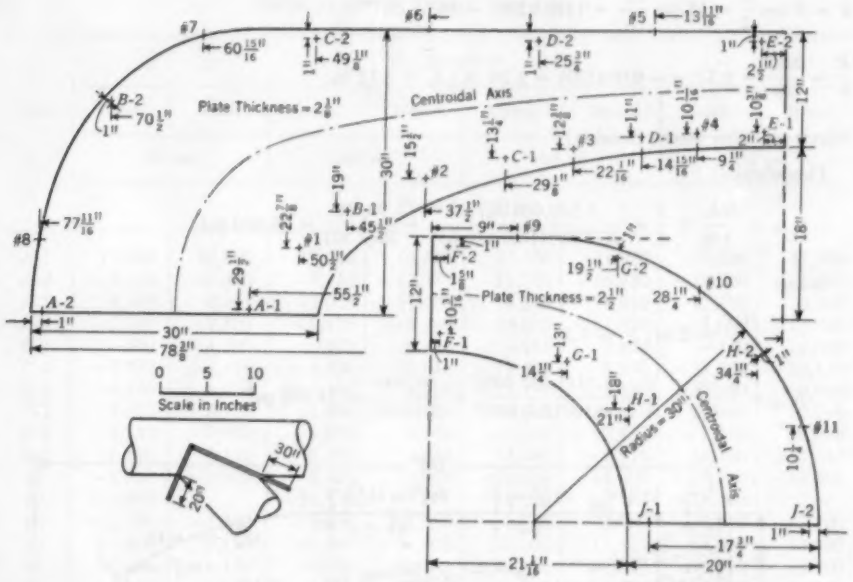
$$W_2 = \frac{p \sin \Delta (L_2)^2}{2} = \frac{385(0.707)(21.06)^2}{2} = 60{,}360 \text{ lb}$$

$$m_1 = \frac{px^3 \sin \Delta}{6} = \frac{385(0.707)x^3}{6} = 45.36x^3$$

x in.	$M = m_1 - xy$ in.-lb	I in.^{4}	$x(\Delta s)$ in.	$\frac{x(\Delta s)}{I}$	$\frac{x(\Delta s)}{I}(m_1 - xy)$	Depth of Plate in.
		$s = 9.5\pm$ in.; $t = 2\frac{1}{8}$ in.				
4.70	5,000— 4.7y	313	44.66	0.143	1,000— 0.67y	12.1
14.1	127,000—14.1y	371	134.0	0.361	46,000— 5.09y	12.8
23.5	588,000—23.5y	486	223.0	0.460	270,000—10.80y	14.0
32.8	1,601,000—32.8y	767	311.5	0.406	650,000—13.32y	16.3
42.0	3,360,000—42.0y	1,416	399.0	0.282	947,000—11.85y	20.0
51.2	6,036,000—51.2y	3,111	487.0	0.157	948,000— 8.04y	26.0
51.38	6,093,000—51.4y	4,875	488.0	0.100	609,000— 5.14y	30.2
51.38	6,093,000—51.4y	4,875	488.0	0.100	609,000— 5.14y	30.2
					4,080,000—60.05y	
		$s = 7.5$ in.; $t = 2.5$ in.				
3.7	2,000+ 3.7y	378	26.25	0.0694	0+ 0.26y	12.2
10.9	59,000+10.9y	559	81.75	0.1465	9,000+ 1.60y	13.9
18.0	264,000+18.0y	886	135.0	0.153	40,000+ 2.75y	16.2
23.06	544,000+23.1y	1,235	173.0	0.1401	76,000+ 3.23y	18.1
23.06	544,000+23.1y	1,519	173.0	0.1139	62,000+ 2.63y	19.4
23.06	544,000+23.1y	1,664	173.0	0.1040	57,000+ 2.40y	20.0
					244,000+12.87y	

4,080,000 − 244,000 = 60.05y + 12.87y; y = 52,600 lb *

* To obtain new value for y for any other pressure, multiply y by new pressure and divide by 385.



Item	Stress*—lb		Item	Stress*—lb	
	Computed	Measured		Computed	Measured
A-1	9,220	10,620	*H*-2	−8,320	−6,710
A-2	−6,520	−5,420	*J*-1	13,300	11,070
B-1	7,540	11,140	*J*-2	−7,470	−7,300
B-2	−5,270	−4,260	#1	9,560	12,250
C-1	−4,600	980	#2	2,460	3,930
C-2	−7,890	−6,450	#3	−7,690	−270
D-1	−2,800	−930	#4	−6,020	780
D-2	7,560	5,900	#5	−6,120	−6,320
E-1	−2,780	−2,860	#6	−5,120	−3,760
E-2	3,210	820	#7	370	210
F-1	2,010	2,850	#8	10,270	8,600
F-2	−2,290	−1,480	#9	−6,320	−5,750
G-1	6,980	10,970	#10	−8,290	−6,780
G-2	−6,610	−5,690	#11	−9,140	−7,050
H-1	8,680	11,920			

* At 335-psi pressure.

Fig. 8. Two-Plate Design (Example 2)—Plan of Stiffener Plates

See italic note below Fig. 7.

$$a = R\cos\frac{\Delta}{2} = 18\cos\frac{45^\circ}{2} = 18(0.9239) = 16.63 \text{ in.}$$

$$\frac{R}{h} = \frac{26.6}{20} = 1.33; e = 0.064(20) = 1.28 \text{ in.}; h_1 = 8.72 \text{ in.}$$

Maximum fiber stress (tension):

Timoshenko:

$$f_{max} = \frac{Mh_1}{Aea} + \frac{T}{A} = \frac{1{,}530{,}000(8.72)}{20(2.50)(1.28)(16.63)} + \frac{98{,}300}{20(2.50)} = 14{,}500 \text{ psi}$$

Seely:

$$\frac{R}{c} = \frac{26.6}{10} = 2.66; K = 1.35$$

$$f_{max} = \frac{KMc}{I} + \frac{T}{A} = \frac{1.35(1{,}530{,}000)}{0.167(2.5)(20)^3} + \frac{98{,}300}{20(2.50)} = 14{,}400 \text{ psi}$$

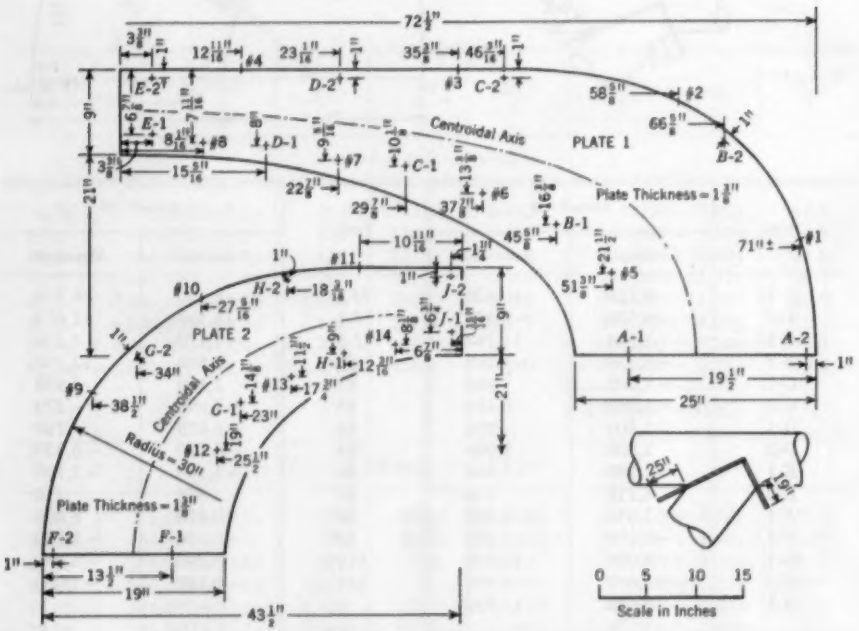


Fig. 9. Two-Plate Design—Plan of Stiffener Plates

Data: $R_B = 21$ *in.;* $R_a = 21$ *in.;* $\Delta = 53$ *deg;* $p = 192$ *psi. See italic note below Fig. 7.*

Table of Stress Values (see Fig. 9)

Item	Stress—*lb*							
	Comp.	Meas.	Comp.	Meas.	Comp.	Meas.	Comp.	Meas.
	192 psi		285 psi		385 psi		510 psi	
				Plate 1				
A-1	11,600	12,000	17,240	16,896	23,350	21,030	29,300	27,500
A-2	−5,600	−7,400	−8,310	−12,288	−11,150	−16,450	−13,900	−21,000
B-1	8,100	8,052	12,025	17,480	16,240	24,010	22,700	26,900
B-2	−5,000	−7,800	−7,470	−9,248	−10,090	−11,590	−14,000	−13,900
C-1	−300	−4,200	−429	6,816	−485	9,840	−140	13,000
C-2	−5,500	−5,350	−8,100	−10,016	−11,000	−9,290	−15,880	−15,140
D-1	−8,130	1,779	−12,100	1,768	−15,700	3,880	−21,780	3,080
D-2	8,070	8,150	11,980	11,984	16,600	16,810	19,900	22,240
E-1	−2,740	5,652	−4,060	7,184	−5,430	10,430	−7,270	6,800
E-2	4,150	2,110	6,190	−1,864	8,330	−1,616	10,160	−1,240
#1	−5,700	−7,050	−8,490	−10,650	−11,020	−14,400	−15,900	−18,000
#2	−9,000		−14,130		−19,100		−25,000	
#3	−2,100	−1,200	−3,157	−2,850	− 4,125	−3,600	−7,210	−2,100
#4	13,010	9,150	19,300	14,070	26,100	17,520	34,800	23,400
#5	6,100	6,150	9,020	10,500	12,210	16,500	16,830	22,200
#6	5,400	8,250	7,930	10,500	10,700	15,150	15,250	20,100
#7	−5,930	2,850	−8,779	1,950	−12,104	3,150	−15,600	5,400
#8	−7,280		−10,800	3,150	−14,600	4,950	−19,300	6,450
				Plate 2				
F-1	7,940	16,260	11,520	20,968	15,910	31,590	+18,730	41,210
F-2	−8,800	−9,940	−12,480	−15,728	−17,640	−20,080	−18,570	−26,900
G-1	5,280	15,500	7,810	20,232	10,540	27,260	+12,740	35,980
G-2	−8,940	−8,970	−12,690	−13,888	−17,940	−18,710	−20,400	−24,000
H-1	7,140	10,740	10,560	14,032	14,300	19,900	+19,300	25,000
H-2	−9,840	−6,520	−14,578	−9,832	−19,690		−26,500	
J-1	390	1,200	977	2,472	1,310	−2,850	440	2,010
J-2	−530		−1,315		−1,765		−1,580	
#9	−9,250	−9,600	−13,130	−13,800	−18,480	−19,200	−21,430	−21,800
#10	−8,050	−8,700	−10,270	−13,200	−16,070	−17,250	−19,130	−21,900
#11	−9,600	−6,450	−14,300	−9,600	−19,300	−13,050	−25,600	−17,250
#12	9,540	13,200	13,700	20,550	19,060	28,350	23,150	37,650
#13	8,600	12,450	12,700	16,500	17,220	23,820	11,550	30,720
#14	5,600	9,000	8,053	12,150	11,000	17,100	−14,500	22,050

Three-Plate Design

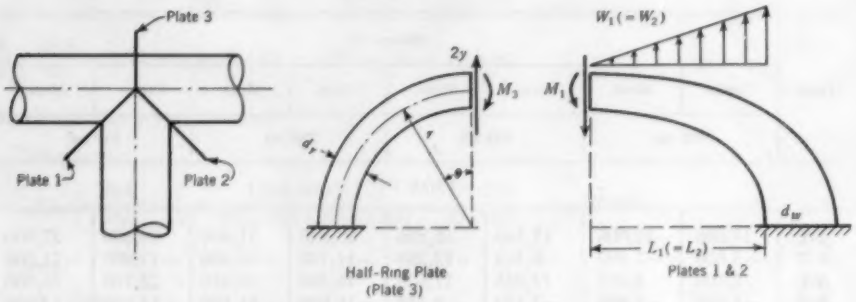


Fig. 10. Three-Plate Design (Example 1)

Data: $R_B = 12$ *in.*; $R_s = 12$ *in.*; $\Delta = 90$ *deg*; $p = 350$ *psi*; *pipe shell thickness,* $\frac{3}{8}$ *in.*; *depth at crown, 5 in.*

Design Assumptions

1. Rotation of tangent to centroidal axis of half-ring plate at crown equals zero; that is, this plate is free but guided at the crown.
2. Moments at crown of Plates 1 and 2 equal $\frac{\sqrt{2}}{2}$ times the moment at crown of half-ring plate, from vector considerations.

Method of Solution

The three unknown quantities, M_1, M_3, and y, require three equations for solution. Two of these equations are obtained from the two design assumptions, and the third is derived from the condition that the deflections of all three plates at the crown are equal.

Solution

For half-ring plate:

$$\alpha = \text{rotation at crown} = \int \frac{Mm\,ds}{EI}$$

$$= \int \frac{(-M_3 + 2yx)(-1)\,ds}{EI} = 0$$

$$\int_0^{\frac{\pi}{2}} \frac{(-M_3 + 2yr\sin\theta)(-1)r\,d\theta}{EI}$$

$$= \int_0^{\frac{\pi}{2}} \frac{(M_3 r - 2yr^2\sin\theta)\,d\theta}{EI} = 0$$

$$(M_3\theta - 2yr^2\cos\theta)_0^{\frac{\pi}{2}} = 0$$

$$M_3\left(\frac{\pi}{2}\right) - 2yr(1) = 0$$

$$M_3 = \frac{4yr}{\pi} \quad \text{(Eq 1)}$$

δ_3 = deflection at crown $= \int \frac{Mm\,ds}{EI}$

$$= \int \frac{(-M_3 + 2yx)x\,ds}{EI}$$

$$= \int_0^{\frac{\pi}{2}} \frac{(-M_3 + 2yr\sin\theta)(r\sin\theta)r\,d\theta}{EI}$$

$$= \frac{-4yr}{\pi}\int_0^{\frac{\pi}{2}} \frac{r\sin\theta\,d\theta}{EI}$$

$$+ 2y\int_0^{\frac{\pi}{2}} \frac{r^2\sin^2\theta\,d\theta}{EI}$$

$$= \frac{-4yr^3}{\pi EI} + \frac{\pi yr^3}{2EI} = \frac{yr^3}{EI}\left(\frac{\pi}{2} - \frac{4}{\pi}\right)$$

$$= \frac{yr^3}{EI}(0.2976)$$

Computation of Maximum Stresses

Half-ring plate:

$$\frac{r}{h} = \frac{14.75}{4.5} = 3.28$$

$e = 0.1151$ in.

TABLE 4

Three-Plate Design Calculations (Example 1)

$$L_1 = \frac{R}{\sin\frac{\Delta}{2}} = \frac{12.0}{0.7071} = 16.97 \text{ in.}; \quad L_1 = L_2$$

$$W_1 = \frac{p}{2} L_1^2 \sin\Delta = \frac{350}{2}(16.97)(0.7071) = 50{,}400 \text{ lb}$$

$$m = \frac{px^2 \sin\Delta}{6} = \frac{350x^2}{6} = 58.33x^2$$

$$M_1 = \frac{\sqrt{2}M_2}{2} = \frac{\sqrt{2}(4yr)}{2\pi} = \frac{2\sqrt{2}(14.75)y}{\pi} = 13.28y \quad \text{(Eq 2)}$$

Plates 1 and 2 (t = 1.5 in.; Δs = 4.17 in.; d_w = 9 in.):

x in.	$M = m - xy - M_1$ in.-lb	Plate Depth in.	I in.^{4}	$x(\Delta s)$	$\frac{x(\Delta s)}{I}$	$\frac{Mx(\Delta s)}{I}$
2.10	540 − 2.10y − 13.28y	5.0	15.62	8.76	0.561	300 − 8.63y
6.22	14,037 − 6.22y − 13.28y	5.15	17.08	25.94	1.519	21,300 − 29.62y
10.27	63,175 − 10.27y − 13.28y	5.5	20.80	42.83	2.059	130,100 − 48.49y
14.09	163.157 − 14.09y − 13.28y	6.1	28.38	58.76	2.070	337,700 − 56.66y
17.50	311,976 − 17.50y − 13.28y	7.0	42.88	72.97	1.702	531,000 − 52.39y
20.32	366,912 − 18.59y − 13.28y	8.0	64.00	77.52	1.211	444,300 − 38.59y
22.06	366,912 − 18.59y − 13.28y	8.9	88.12	77.52	0.880	322,900 − 28.04y
						1,787,600 − 262.42y = $E\delta_1$

For half ring:

$$\delta_1 = \frac{0.2976yr^3}{EI}; \quad \delta_1 = \delta_2 \quad \text{(Eq 3)}$$

$$1{,}787{,}600 - 262.42y = \frac{0.2976yr^3}{I} = \frac{0.2976y(3{,}209)}{11.39} = 83.85y$$

$$y = 5{,}162 \text{ lb}; \quad M_2 = \frac{4r(5{,}162)}{\pi} = 96{,}940 \text{ in.-lb}; \quad M_1 = 68{,}550 \text{ in.-lb}$$

Maximum moment:

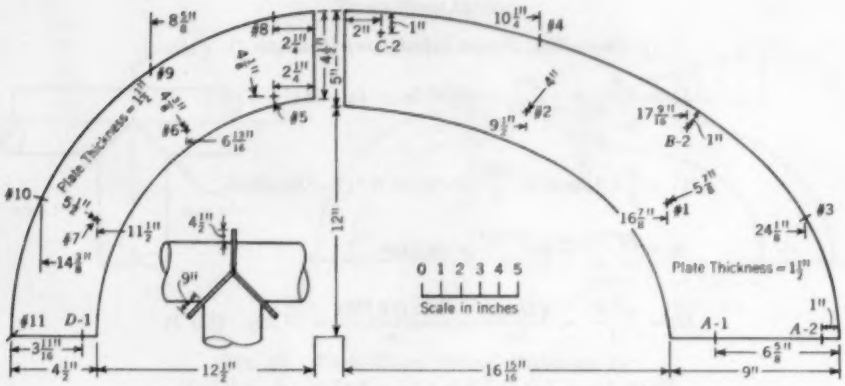
$$2yr - M_2 = 2(5{,}162)(14.75) - 96{,}940 = 55{,}340 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{Mh_1}{Aea} + \frac{T}{A} = \frac{55{,}340(2.25 - 0.115)}{1.5(4.5)(0.1151)(12.5)} + \frac{2(5{,}162)}{1.5(4.5)} = +12{,}170 + 1{,}530 = +13{,}700 \text{ psi}$$

Maximum compressive stress:

$$\frac{Mh_2}{Aec} + \frac{T}{A} = \frac{55{,}340(2.25 + 0.115)}{1.5(4.5)(0.1151)(17.0)} + \frac{2(5{,}162)}{1.5(4.5)} = -9{,}910 + 1{,}530 = -8{,}380 \text{ psi}$$



Item	Stress*—lb		Item	Stress*—lb	
	Computed	Measured		Computed	Measured
A-1	7,360	6,840	#4	8,380	6,150
A-2	−3,160	−3,260	#5	−8,860	−8,850
B-2	−2,080	−730	#6	−20	−1,650
C-2	7,780	5,090	#7	4,000	6,510
D-1	8,610	11,780	#8	12,360	11,350
#1	6,950	7,350	#9	2,750	3,600
#2	−2,030	3,380	#10	−6,000	−3,450
#3	−4,400	−3,450	#11	−8,380	−6,900

* At 350-psi pressure.

Fig. 11. Three-Plate Design (Example 1)—Plan of Stiffener Plates

See italic note below Fig. 7.

Horseshoe plate (Plates 1 and 2):

$$a = R \sin \frac{\Delta}{2} = 12.0(0.7071) = 8.49 \text{ in.}$$

$$\frac{r}{h} = \frac{8.485 + 4.5}{9.0} = 1.443; \; e = 0.536 \text{ in.}$$

Maximum moment:

$$W_1\left(\frac{L_1}{3} + 0.72^* + \frac{d_w}{10}\right) - y\left(L_1 + 0.72^* + \frac{d_w}{10}\right) - \frac{\sqrt{2}}{2} M_2$$

$$= 50{,}400\left(\frac{16.97}{3} + 0.72 + \frac{9}{10}\right) - 5{,}162\left(16.97 + 0.72 + \frac{9}{10}\right) - \frac{\sqrt{2}}{2}(96{,}940)$$

$$= 366{,}760 - 95{,}920 - 68{,}550 = 202{,}290 \text{ in.-lb}$$

* This additional distance was made necessary in the fabrication of the wye by the requirements for fitting the plates together properly.

Maximum tensile stress:

$$\frac{Mh_1}{Aea} + \frac{T}{A} = \frac{202{,}290(4.5 - 0.536)}{1.5(9.0)(0.536)(8.49)} + \frac{50{,}400 - 5{,}160}{9.0(1.5)}$$

$$= +13{,}050 + 3{,}350 = +16{,}400 \text{ psi}$$

Maximum compressive stress:

$$\frac{Mh_2}{Aec} + \frac{T}{A} = \frac{202{,}290(4.5 + 0.536)}{1.5(9.0)(0.536)(17.49)} + 3{,}350 = -8{,}050 + 3{,}350 = -4{,}700 \text{ psi}$$

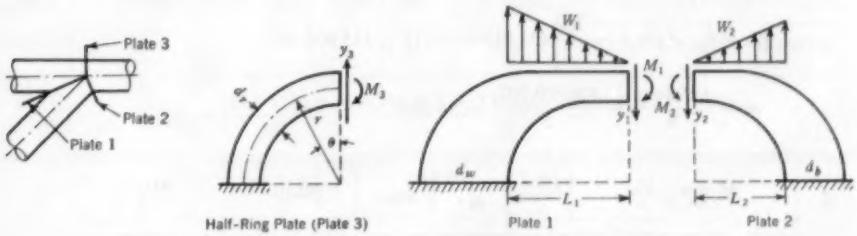


Fig. 12. Three-Plate Design (Example 2)

Data: $R_B = 30.5$; $R_a = 30.5$; $\Delta = 45$ deg; $p = 300$ psi; pipe shell thickness, $\frac{7}{16}$ in.; depth at crown, 12 in.

Design Assumptions

1. Moment at crown of Plate 1 (M_1) equals zero.
2. Rotation of tangents to centroidal axes of Plates 2 and 3 at crown equals zero; that is, these plates are free but guided at the crown.

Method of Solution

The five unknown quantities—y_1, y_2, y_3, M_2, and M_3—require five conditions for solution. They are:

1. Rotation of tangent to centroidal axis of Plate 2 equals zero at crown.
2. Rotation of tangent to centroidal axis of Plate 3 equals zero at crown.
3. Deflections of Plates 1 and 2 at crown are equal.
4. Deflections of Plates 2 and 3 at crown are equal.
5. Shear at end of Plate 3 equals sum of shears at ends of Plates 1 and 2.

The method of virtual work is used to obtain equations based on the first four conditions.

Solution

For half-ring plate:

$$\alpha = \text{rotation at crown} = \int \frac{Mm\,ds}{EI}$$

$$= \int \frac{(-M_2 + y_3 x)(-1)\,ds}{EI} = 0$$

$$\int_0^{\frac{\pi}{2}} (-M_2 + y_3 r \sin\theta)(-1) r\,d\theta$$

$$= \int_0^{\frac{\pi}{2}} M_2 r\,d\theta - \int_0^{\frac{\pi}{2}} y_3 r^2 \sin\theta\,d\theta = 0$$

$$(M_2 r\theta)_0^{\frac{\pi}{2}} + (y_3 r^2 \cos\theta)_0^{\frac{\pi}{2}} = 0$$

$$M_2 r \frac{\pi}{2} - y_3 r^2 = 0$$

$$M_2 = \frac{2y_3 r}{\pi} = \frac{2(y_3)(36.875)}{\pi}$$

$$= 23.475 y_3 \qquad \text{(Eq 2)}$$

TABLE 5

Three-Plate Design Calculations (Example 2)

$$L_1 = \frac{R}{\sin\frac{\Delta}{2}} = \frac{30.5}{0.3827} = 79.70 \text{ in.}; \quad L_2 = \frac{R}{\cos\frac{\Delta}{2}} = \frac{30.5}{0.9239} = 33.01 \text{ in.}$$

L_1 (scaled from Plate 1) = 82.72 in.; L_2 (scaled from Plate 2) = 33.53 in.

$$W_1 = \frac{p}{2} L_1^3 \sin\Delta = \frac{300}{2} (79.70)^2(0.7071) = 673{,}700 \text{ lb}$$

$$W_2 = \frac{p}{2} L_2^3 \sin\Delta = \frac{300}{2} (33.01)^3(0.7071) = 115{,}600 \text{ lb}$$

$$m = \frac{px^3 \sin\Delta}{6} = \frac{300x^3(0.707)}{6} = 35.36x^3$$

x in.	$M = m - xy_1$ in.-lb	Plate Depth in.	I in.^{4}	$x(\Delta s)$	$\frac{x(\Delta s)}{I}$	$\frac{Mx(\Delta s)}{I}$
Plate 1 (t = 1 in. (assumed); Δs = 15.1 in.; d_w = 54 in.)						
7.6	15,500 − 7.6y_1	12.0	144	114.8	0.7972	12,400 − 6.06y_1
22.7	413,600 − 22.7y_1	13.2	192	342.8	1.7854	738,400 − 40.53y_1
37.8	1,909,800 − 37.8y_1	15.5	310	570.8	1.8413	3,516,500 − 69.60y_1
52.9	5,234,600 − 52.9y_1	19.4	608	798.8	1.3138	6,877,200 − 69.50y_1
67.7	10,971,800 − 67.7y_1	25.4	1,366	1,022.3	0.7484	8,211,300 − 50.67y_1
82.2	19,584,500 − 82.2y_1	35.5	3,728	1,241.2	0.3329	6,519,700 − 27.36y_1
85.1	21,538,200 − 85.1y_1	48.0	9,216	1,285.0	0.1394	3,002,400 − 11.86y_1
85.1	21,538,200 − 85.1y_1	53.3	12,618	1,285.0	0.1018	2,192,600 − 8.66y_1
						31,070,500 − 284.24y_1
Plate 2 (t = 1 in. (assumed); Δs = 10.7 in.; d_b = 22 in.)						
5.4	5,600 − 5.4y_2 − M_2	12.5	163	57.8	0.3546	2,000 − 1.91y_2 − 0.3546M_2
15.8	139,500 − 15.8y_2 − M_2	15.0	281	169.1	0.6018	84,000 − 9.51y_2 − 0.6018M_2
25.8	607,300 − 25.8y_2 − M_2	17.7	462	276.1	0.5976	362,900 − 15.42y_2 − 0.5976M_2
34.1	1,398,800 − 34.1y_2 − M_2	19.9	657	364.9	0.5554	776,900 − 18.94y_2 − 0.5554M_2
35.2	1,525,900 − 35.2y_2 − M_2	21.2	794	376.6	0.4743	723,700 − 16.70y_2 − 0.4743M_2
35.2	1,525,900 − 35.2y_2 − M_2	22.0	887	376.6	0.4246	647,900 − 14.95y_2 − 0.4246M_2
						2,597,400 − 77.43y_2 − 3.008M_2

Plate 1: $tE\delta_1 = 31{,}070{,}500 - 284.24y_1$

Plate 2: $tE\delta_2 = 2{,}597{,}400 - 77.43y_2 - 3.008M_2$

Plate 3: $tE\delta_3 = 62.88y_2$

TABLE 5 (*contd.*)

Plate 2—Equation for Zero Rotation at Crown

x in.	M in.-lb	Plate Depth in.	I in.^{4}	$\frac{(-1)(\Delta s)}{I}$	$\frac{M(-1)(\Delta s)}{I}$
5.4	$5{,}600 - 5.4y - M_2$	12.5	163	−0.06564	$-368 + 0.354y_2 + 0.0656M_2$
15.8	$139{,}500 - 15.8y - M_2$	15.0	281	−0.03808	$-5{,}312 + 0.602y_2 + 0.0381M_2$
25.8	$607{,}300 - 25.8y - M_2$	17.7	462	−0.02316	$-14{,}065 + 0.598y_2 + 0.0232M_2$
34.1	$1{,}398{,}800 - 34.1y - M_2$	19.9	657	−0.01629	$-22{,}786 + 0.555y_2 + 0.0163M_2$
35.2	$1{,}525{,}900 - 35.2y - M_2$	21.2	794	−0.01348	$-20{,}569 + 0.474y_2 + 0.0135M_2$
35.2	$1{,}525{,}900 - 35.2y - M_2$	22.0	887	−0.01206	$-18{,}402 + 0.425y_2 + 0.0121M_2$
					$-81{,}502 + 3.008y_2 + 0.1687M_2$

$$\alpha = \frac{M(-1)(\Delta s)}{EI} = \frac{-81{,}502 + 3.008y_2 + 0.1687M_2}{E} = 0$$

$$M_2 = \frac{81{,}502 - 3.008y_2}{0.1687} = 483{,}100 - 17.83y_2 \quad \text{(Eq 1)}$$

δ_3 = deflection of crown $= \int \frac{Mm\,ds}{EI} = \int \frac{(-M_2 + y_3 x)x\,ds}{EI}$

$$= \frac{1}{EI}\int_0^{\frac{\pi}{2}} (-M_2 r \sin\theta) r\,d\theta + \frac{1}{EI}\int_0^{\frac{\pi}{2}} (y_3 r^2 \sin^2\theta) r\,d\theta$$

$$= \frac{1}{EI}(M_2 r^2 \cos\theta)^{\frac{\pi}{2}} + \frac{1}{EI}\left[y_3 r^3\left(\frac{\theta}{2} - \frac{\sin\theta\cos\theta}{2}\right)\right]^{\frac{\pi}{2}}$$

$$= \frac{1}{EI}\left(-M_2 r^2 + \frac{\pi r^3 y_3}{4}\right) = \frac{1}{EI}\left(\frac{-2y_3 r^2}{\pi} + \frac{\pi r^3 y_3}{4}\right) = \frac{r^3 y_3}{EI}\left(\frac{\pi}{4} - \frac{2}{\pi}\right)$$

$$= \frac{0.1488 r^3 y_3}{EI} = \frac{0.1488(36.875)^3 y_3(12)}{E(1.0)(11.25)^3} = \frac{62.88y_4}{E}$$

NOTE: In solving simultaneous equations for shears and moments at the ends of plates (crown), all plates are taken to be 1 in. thick. As I occurs in the denominator of all three deflection equations, it does not matter what thickness is used, so long as all three thicknesses are equal. When solving for stresses in the plates, the actual thicknesses will be used.

Plate 1:

$$EI\delta_1 = 31{,}070{,}500 - 284.24y_1$$

Plate 2:

$$EI\delta_2 = 2{,}597{,}400 - 77.43y_2 - 3.008M_2$$
$$= 2{,}597{,}400 - 77.43y_2 - 1{,}453{,}200 + 53.63y_2 = 1{,}144{,}200 - 23.80y_2$$

Plate 3:

$$EI\delta_3 = 62.88y_3$$
$$31{,}070{,}500 - 284.24y_1 = 1{,}144{,}200 - 23.80y_3 \quad \text{(Eq 3)}$$

$$284.24y_1 - 23.80y_2 - 29{,}926{,}300 = 0$$

$$1{,}144{,}200 - 23.80y_2 = 62.88y_3 \quad \text{(Eq 4)}$$

$$y_3 = y_1 + y_2 \quad \text{(Eq 5)}$$

$$\begin{cases} 62.88y_1 + 86.68y_2 - 1{,}144{,}200 = 0 \\ -(62.88y_1 - 5.26y_2 - 6{,}620{,}300 = 0) \end{cases}$$

$$91.94y_2 + 5{,}476{,}100 = 0$$

$y_2 = -59{,}560$ lb; $y_3 = +40{,}740$ lb; $y_1 = 100{,}300$ lb

$M_3 = 23.475y_3 = 956{,}400$ in.-lb

$M_2 = 483{,}100 - 17.83(-59{,}560) = 1{,}545{,}000$ in.-lb

Computation of Maximum Stresses

Plate 1:

$$a = R \sin \frac{\Delta}{2} = 30.5(0.3827) = 11.68 \text{ in.}$$

$$\frac{r}{h} = \frac{11.68 + 27.0}{54.00} = 0.716; c = 7.098 \text{ in.}$$

Maximum moment:

$$W_1\left(\frac{L_1}{3} + 3.02^* + \frac{d_w}{10}\right) - y_1\left(L_1 + 3.02 + \frac{d_w}{10}\right)$$

$$= 673{,}700\left(\frac{79.70}{3} + 3.02 + \frac{54.0}{10}\right) - 100{,}300\left(79.70 + 3.02 + \frac{54.0}{10}\right)$$

$$= 23{,}573{,}000 - 8{,}838{,}000 = 14{,}735{,}000 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{Mh_1}{Aea} + \frac{T}{A} = \frac{14{,}735{,}000(27.0 - 7.1)}{54.0(2.5)(7.098)(11.68)} + \frac{673{,}700 - 100{,}300}{54.0(2.5)} = 26{,}200 + 4{,}250 = 30{,}450 \text{ psi}$$

Maximum compressive stress:

$$\frac{Mh_2}{Aec\dagger} + \frac{T}{A} = \frac{-14{,}735{,}000(27.0 + 7.1)}{54.0(2.5)(7.098)(42.5)} + 4{,}250 = -12{,}340 + 4{,}250 = -8{,}090 \text{ psi}$$

Plate 2:

$$\frac{r}{c} = \frac{35.0\ddagger}{11.0} = 3.18; K_i = 1.28; K_o = 0.799$$

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* This additional distance was made necessary in the fabrication of the wye by the requirements for fitting the plates together properly.

† c is the radius of curvature to the outside fibers (Timoshenko), scaled in this case (see Fig. 22, page 626) owing to extremely nonconcentric curves.

‡ Scaled.

Maximum moment:

$$W_2\left(\frac{L_2}{3}+0.52^*+\frac{d_b}{10}\right)-y_2\left(L_2+0.52+\frac{d_b}{10}\right)-M_2$$

$$=115{,}600\left(\frac{33.01}{3}+0.52+\frac{22}{10}\right)+59{,}560\left(33.01+0.52+\frac{22.0}{10}\right)-1{,}545{,}000$$

$$=1{,}586{,}000+2{,}127{,}500-1{,}545{,}000=2{,}168{,}500 \text{ in.-lb}$$

Maximum tensile stress (Seely):

$$\frac{KMc}{I}+\frac{T}{A}=\frac{1.28(2{,}168{,}500)(6)}{2.5(22.0)^3}+\frac{115{,}600+59{,}560}{22.0(2.5)}=13{,}770+3{,}180=+16{,}950 \text{ psi}$$

Maximum compressive stress:

$$\frac{-0.799(2{,}168{,}500)(6)}{2.5(22.0)^3}+3{,}180=-8{,}600+3{,}180=-5{,}420 \text{ psi}$$

Plate 3 (half-ring):

$$\frac{r}{c}=\frac{36.875}{5.625}=6.56;\ K_i=1.109;\ K_o=0.910$$

Maximum moment:

$$y_3(r)-M_3=40{,}740(36.875)-956{,}400=545{,}900 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{K_iMc}{I}+\frac{T}{A}=\frac{1{,}109(545{,}900)(6)}{2.5(11.25)}+\frac{40{,}740}{2.5(11.25)}=11{,}480+1{,}450=+12{,}930 \text{ psi}$$

Maximum compressive stress:

$$\frac{K_oMc}{I}+\frac{T}{A}=\frac{-0.910(545{,}900)(6)}{2.5(11.25)}+1{,}450=-9{,}420+1{,}450=-7{,}970 \text{ psi}$$

TABLE 6

Three-Plate Design (Example 2)—Comparison of Results Obtained Using Three Different Assumptions

(a) All plates free but guided at crown ($s_1 = s_2 = s_3 = 0$).
(b) Plate 3 free but guided at crown, others simply supported.
(c) Plates 2 and 3 free but guided at crown, Plate 1 simply supported.

Plate	Location	Stress—*psi*			
		Computed			Measured
		(a)	(b)	(c)	
1	outside of crotch	− 5,500	− 7,050	− 8,090	− 5,850
1	inside of crotch	+26,900	+28,400	+30,450	+34,300
2	outside of crotch	− 6,600	− 6,780	− 5,610	− 6,150
2	inside of crotch	+19,500	+16,950	+16,950	+16,700
3	outside of crotch	−10,100	−13,680	− 7,930	−10,350
3	inside of crotch	+16,490	+22,330	+12,970	+15,750
3	outside of crown	+18,960	+25,700	+15,800	+16,200
3	inside of crown	−23,300	−31,500	−19,300	−29,100

From these and other comparisons it may be seen that the best results are obtained when using the design assumptions in which Plates 2 and 3 are considered free but guided (that is, slopes of tangents are zero at crown) and Plate 1 is considered simply supported. The justification for these assumptions, aside from better agreement with test results, is that Plates 2 and 3 are almost in line with one another and provide the fixity needed by each plate for zero rotation at the crown. Plate 1, however, is much longer than either of the others, has less rigidity, and has no opposing plate to restrain its rotation at the crown. Therefore, even though it is not completely simply supported, its degree of fixity is relatively low, and for practical purposes is considered zero.

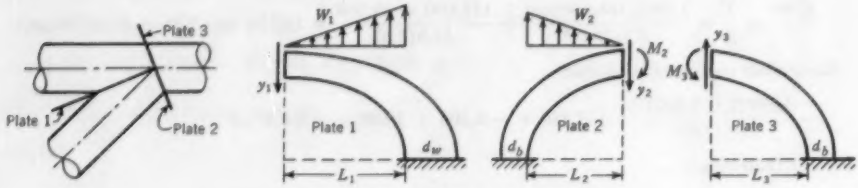


Fig. 13. Three-Plate Design (Example 3)

Data: $R_B = 18$ *in.;* $R_s = 18$ *in.;* $\Delta = 45$ *deg;* $p = 335$ *psi; pipe shell thickness,* $\frac{3}{8}$ *in.; depth at crown, 12 in.*

Design Assumptions

1. Moment at crown of Plate 1 (M_1) equals zero.
2. Rotation of tangents to centroidal axes of Plates 2 and 3 at crown equals zero; that is, these plates are free but guided at the crown.

Method of Solution

The method of solution is similar to that used in Example 2, the main difference being that Plate 3 is elliptical, not circular. In the following analysis, five unknowns will be considered: y_1, y_2, and y_3 (the shears at the ends of Plates 1, 2, and 3, respectively); and M_2 and M_3 (the moments at the crown ends of Plates 2 and 3). M_1 is assumed to be zero, while M_2 and M_3 are such that the tangents to the plates at the crown remain at zero slope under load.

These unknowns are solved under the following five conditions:

1. Slope of tangent to Plate 2 at crown equals zero.
2. Slope of tangent to Plate 3 at crown equals zero.
3. $y_3 = y_1 + y_2$
4. Deflection at crown of Plate 1 equals that of Plate 2.
5. Deflection at crown of Plate 2 equals that of Plate 3.

The equations for virtual work are applied to conditions 1, 2, 4, and 5:

Eq 1:

$$M_2 = 206{,}000 - 15.17y_2$$

Eq 2:

$$M_3 = 16.47y_3$$

Eq 4:

$$4{,}022{,}500 - 64.71y_1 = 1{,}079{,}400 - 72.61y_2 - 4.028M_2$$

$$2{,}943{,}100 - 64.71y_1 = -72.61y_2 - 4.028(206{,}000 - 15.17y_2)$$

$$= -72.61y_2 - 829{,}800 + 61.10y_2$$

$$3{,}772{,}900 - 64.71y_1 + 11.51y_2 = 0$$

Eq 5:

$$1{,}079{,}400 - 72.61y_2 - 4.028M_2 = 91.44y_3 - 4.500M_3$$

$$1{,}079{,}400 - 72.61y_2 - 4.028(206{,}000 - 15.17y_2) = 91.44y_3 - 4.500(16.47y_3)$$

TABLE 7

Three-Plate Design Calculations (Example 3)

$$L_1 = \frac{R}{\sin\frac{\Delta}{2}} = \frac{18.0}{0.3827} = 47.04 \text{ in.}; \quad L_2 = \frac{R}{\cos\frac{\Delta}{2}} = \frac{18.0}{0.9239} = 19.48 \text{ in.}$$

L_1 (scaled from Plate 1) = 49.5 in.; L_2 (scaled from Plate 2) = 19.75 in.

$$W_1 = \frac{pL_1^2 \sin\Delta}{2} = \frac{335(47.04)^2(0.7071)}{2} = 262{,}000 \text{ lb}$$

$$W_2 = \frac{pL_2^2 \sin\Delta}{2} = \frac{335(19.48)(0.7071)}{2} = 44{,}960 \text{ lb}$$

$$m = \frac{px^2 \sin\Delta}{6} = \frac{335x^2(0.7071)}{6} = 39.48x^2$$

x in.	M in.-lb	Plate Depth in.	I in.^{4}	$x(\Delta s)$	$\frac{x(\Delta s)}{I}$	$\frac{Mx(\Delta s)}{I}$
			Plate 1 (t_1 = 2 in.; Δs = 9.48 in.; d_w = 30 in.)			
4.7	$4{,}100 - 4.7y_1$	12.4	318	44.6	0.1403	$600 - 0.66y_1$
14.2	$113{,}000 - 14.2y_1$	13.0	366	134.6	0.3678	$41{,}600 - 5.22y_1$
23.7	$525{,}600 - 23.7y_1$	14.3	487	224.7	0.4614	$242{,}500 - 10.94y_1$
33.1	$1{,}431{,}700 - 33.1y_1$	16.5	749	313.8	0.4190	$599{,}900 - 13.87y_1$
42.5	$3{,}030{,}700 - 42.5y_1$	20.2	1,374	402.9	0.2932	$888{,}600 - 12.46y_1$
51.6	$5{,}302{,}900 - 51.6y_1$	25.2	2,667	489.2	0.1834	$972{,}600 - 9.46y_1$
52.5*	$5{,}538{,}700 - 52.5y_1$	29.2	4,150	497.7	0.1199	$664{,}100 - 6.29y_1$
52.5	$5{,}538{,}700 - 52.5y_1$	30.0	4,500	497.7	0.1106	$612{,}600 - 5.81y_1$
						$4{,}022{,}500 - 64.71y_1$
			Plate 2 (t_2 = 1 in.; Δs = 6.52 in.; d_b = 12 in.)			
3.2	$1{,}300 - 3.2y_2 - M_2$	12.3	155.1	20.9	0.1348	$200 - 0.43y_2 - 0.135M_2$
9.5	$33{,}800 - 9.5y_2 - M_2$	12.2	151.3	61.9	0.4091	$13{,}800 - 3.89y_2 - 0.409M_2$
15.3	$141{,}400 - 15.3y_2 - M_2$	12.1	147.6	99.8	0.6762	$95{,}600 - 10.35y_2 - 0.676M_2$
20.1	$319{,}700 - 20.1y_2 - M_2$	12.0	144.0	131.0	0.9097	$290{,}800 - 18.28y_2 - 0.910M_2$
20.95	$357{,}900 - 20.9y_2 - M_2$	12.0	144.0	136.6	0.9486	$339{,}500 - 19.83y_2 - 0.949M_2$
20.95	$357{,}900 - 20.9y_3 - M_2$	12.0	144.0	136.6	0.9486	$339{,}500 - 19.83y_2 - 0.949M_2$
						$1{,}079{,}400 - 72.61y_2 - 4.028M_2$
			Plate 3 (t_3 = 1 in.; Δs = 6.56 in.; d_b = 12 in.)			
3.3	$0 + 3.3y_3 - M_3$	12.0	144.0	21.6	0.1500	$+ 0.50y_3 - 0.150M_3$
9.7	$0 + 9.7y_3 - M_3$	12.0	144.0	63.6	0.4417	$+ 4.28y_3 - 0.442M_3$
15.5	$0 + 15.5y_3 - M_3$	12.0	144.0	101.7	0.7062	$+10.95y_3 - 0.706M_3$
20.5	$0 + 20.5y_3 - M_3$	12.0	144.0	134.5	0.9340	$+19.15y_3 - 0.934M_3$
24.0	$0 + 24.0y_3 - M_3$	12.0	144.0	157.4	1.0933	$+26.24y_3 - 1.093M_3$
25.8	$0 + 25.8y_3 - M_3$	12.0	144.0	169.2	1.1753	$+30.32y_3 - 1.175M_3$
						$+91.44y_3 - 4.500M_3$

* Maximum x distance is to point one-tenth depth of plate from inside edge.

TABLE 7 (*contd.*)

Equation for Zero Rotation at Crown

x in.	$M = m - xy_2 - M_2$ in.-lb	Plate Depth in.	I in.^{4}	$\frac{(-1)(\Delta s)}{I}$	$\frac{M(-1)(\Delta s)}{I}$
		Plate 2			
3.2	$1{,}300 - 3.2y_2 - M_2$	12.3	155.1	−0.04204	$-50 + 0.135y_2 + 0.0420M_2$
9.5	$33{,}800 - 9.5y_2 - M_2$	12.2	151.3	−0.04309	$-1{,}460 + 0.409y_2 + 0.0431M_2$
15.3	$141{,}400 - 15.3y_2 - M_2$	12.1	147.6	−0.04417	$-6{,}250 + 0.676y_2 + 0.0442M_2$
20.1	$319{,}700 - 20.1y_2 - M_2$	12.0	144.0	−0.04528	$-14{,}480 + 0.910y_2 + 0.0453M_2$
20.95	$357{,}900 - 20.9y_2 - M_2$	12.0	144.0	−0.04528	$-16{,}200 + 0.946y_2 + 0.0453M_2$
20.95	$357{,}900 - 20.9y_2 - M_2$	12.0	144.0	−0.04528	$-16{,}200 + 0.946y_2 + 0.0453M_2$
					$-54{,}640 + 4.022y_2 + 0.2652M_2$
		Plate 3			
3.3	$+3.3y_3 - M_3$	12.0	144.0	−0.04555	$-0.150y_3 + 0.0455M_3$
9.7	$+9.7y_3 - M_3$	12.0	144.0	−0.04556	$-0.442y_3 + 0.0456M_3$
15.5	$+15.5y_3 - M_3$	12.0	144.0	−0.04555	$-0.706y_3 + 0.0455M_3$
20.5	$+20.5y_3 - M_3$	12.0	144.0	−0.04556	$-0.934y_3 + 0.0456M_3$
24.0	$+24.0y_3 - M_3$	12.0	144.0	−0.04555	$-1.093y_3 + 0.0455M_3$
25.8	$+25.8y_3 - M_3$	12.0	144.0	−0.04556	$-1.175y_3 + 0.0456M_3$
					$-4.500y_3 + 0.2733M_3$

$$\alpha_2 = \Sigma \frac{Mm\Delta s}{EI} = \Sigma \frac{M(-1)\Delta s}{EI} = \frac{-54{,}640 + 4.022y_2 + 0.2652M_2}{E} = 0$$

$$M_2 = \frac{54{,}640 - 4.022y_2}{0.2652} = 206{,}000 - 15.17y_2 \quad \text{(Eq 1)}$$

$$\alpha_3 = \Sigma \frac{Mm\Delta s}{EI} = \frac{M(-1)(\Delta s)}{EI} = \frac{-4.500y_3 + 0.2733M_2}{E} = 0$$

$$M_3 = \frac{4.500y_3}{0.2733} = 16.47y_3 \quad \text{(Eq 2)}$$

$1{,}079{,}400 - 72.61y_2 - 829{,}900 + 61.10y_2$
$= 91.44y_3 - 74.12y_3$
$249{,}600 - 11.51y_2 = 17.32y_3$

Eq 3:

$y_3 = y_1 + y_2$
$249{,}600 - 11.51y_2 = 17.32(y_1 + y_2)$

$$\begin{cases} 249{,}600 - 28.83y_2 - 17.32y_1 = 0 \\ -1{,}009{,}800 - 3.08y_2 + 17.32y_1 = 0 \end{cases}$$

$$-760{,}200 - 31.91y_2 = 0$$

$y_2 = -23{,}820$ lb

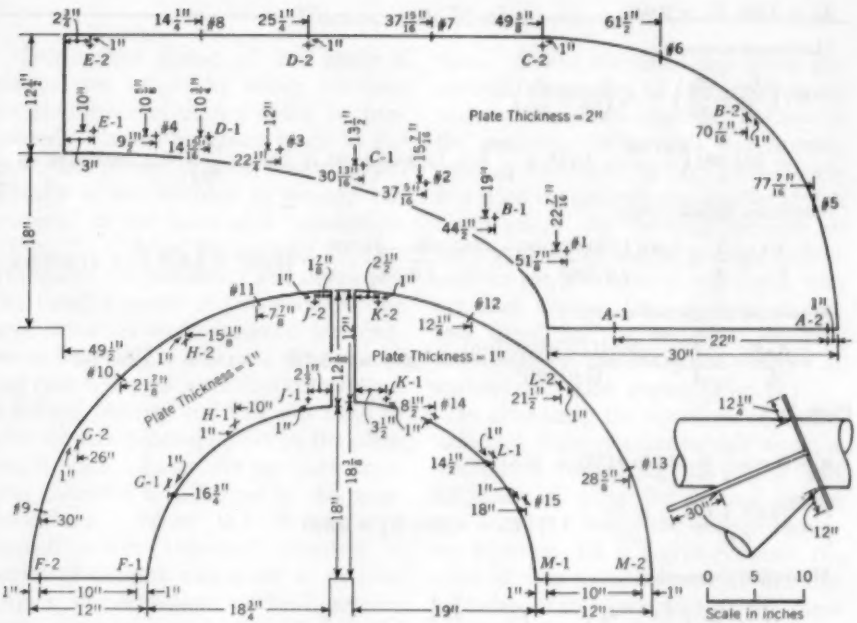
$y_1 = +54{,}060$ lb
$y_3 = +30{,}240$ lb
$M_2 = 567{,}400$ in.-lb
$M_3 = 498{,}000$ in.-lb

Computation of Maximum Stresses

Plate 1:

$$a = R \sin \frac{\Delta}{2} = 18.0(0.3827) = 6.89 \text{ in.}$$

$$\frac{r}{c} = \frac{6.89 + 15.0}{15.0} = \frac{21.89}{15.0} = 1.459$$



Item	Stress*—lb		Item	Stress*—lb	
	Computed	Measured		Computed	Measured
A-1	6,950	9,310	*L*-1	4,710	4,100
A-2	−2,340	−3,730	*L*-2	−630	−800
B-1	8,000	10,130	*M*-1	13,760	13,550
B-2	−3,440	−4,060	*M*-2	−6,400	−6,220
C-1	−3,890	−5,180	#1	7,760	5,460
C-2	−5,570	−3,190	#2	1,160	3,750
D-1	−7,870	−5,580	#3	−8,450	−1,290
D-2	9,250	8,380	#4	−5,980	300
E-1	−1,630	−4,390	#5	−2,740	−4,800
E-2	1,990	2,440	#6	−3,920	−4,350
F-1	14,500	12,500	#7	1,520	2,400
F-2	−3,000	−4,420	#8	11,360	8,550
G-1	14,070	10,900	#9	−4,360	−3,810
G-2	−4,000	−420	#10	−800	2,460
H-1	−4,520	4,800	#11	13,950	9,210
H-2	6,300	7,010	#12	7,220	6,450
J-1	−14,310	−5,150	#13	−5,840	−4,590
J-2	17,700	11,800	#14	−5,530	810
K-1	−12,180	−12,870	#15	10,380	12,150
K-2	13,110	9,850			

* At 335-psi pressure.

Fig. 14. Three-Plate Design (Example 3)—Plan of Stiffener Plates

See italic note below Fig. 7.

$K_i = 2.00$; $K_o = 0.641$

Maximum moment:

$$W_1\left(\frac{L_1}{3} + \frac{d_w}{10}\right) - y_1\left(L_1 + \frac{d_w}{10}\right)$$

$$= 262{,}000\left(\frac{47.04}{3} + 2.46^* + \frac{30}{10}\right) - 54{,}060\left(47.04 + 2.46 + \frac{10}{30}\right) = 2{,}700{,}000 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{K_i Mc}{I} + \frac{T}{A} = \frac{2.00(2{,}700{,}000)(6)}{2.0(30)^3} + \frac{262{,}000 - 54{,}000}{2.0(30)} = 18{,}000 + 3{,}470 = +21{,}470 \text{ psi}$$

Maximum compressive stress:

$$\frac{-K_o Mc}{I} + \frac{T}{A} = \frac{-0.641(2{,}700{,}000)(6)}{(2.0)(30)^3} + 3{,}470 = -5{,}770 + 3{,}470 = -2{,}300 \text{ psi}$$

Plate 2:

$$a = R\cos\frac{\Delta}{2} = 18.0(0.9239) = 16.63 \text{ in.}$$

$$\frac{r}{c} = \frac{16.63 + 6.0}{6.0} = \frac{22.63}{6.0} = 3.77; K_i = 1.220; K_o = 0.840$$

Maximum moment:

$$-M_2 + W_2\left(\frac{L_2}{3} + \frac{d_b}{10}\right) - y_2\left(L_2 + \frac{d_b}{10}\right)$$

$$= -567{,}400 + 44{,}960\left(\frac{19.48}{3} + 0.27^* + \frac{12}{10}\right) - 23{,}820\left(19.75 + \frac{12}{10}\right) = 281{,}400 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{K_i Mc}{I} + \frac{T}{A} = \frac{1.220(281{,}400)(6)}{1.0(12)^3} + \frac{44.960 + 23{,}820}{1.0(12.0)} = 14{,}300 + 5{,}730 = 20{,}030 \text{ psi}$$

Maximum compressive stress:

$$\frac{-K_o Mc}{I} + \frac{T}{A} = \frac{-0.840(281{,}400)(6)}{1.0(12)^3} + 5{,}730 = -9{,}850 + 5{,}730 = -4{,}120 \text{ psi}$$

Plate 3

$K_i = 1.220$; $K_o = 0.840$

Maximum moment:

$$-M_3 + y_3\left(L_3 + \frac{d_b}{2}\right) = -498{,}000 + 30{,}240\left(19.48 + 0.52^* + \frac{12.0}{2}\right) = 288{,}200 \text{ in.-lb}$$

Maximum tensile stress:

$$\frac{K_i Mc}{I} + \frac{T}{A} = \frac{1.220(288{,}200)(6)}{1.0(12)^3} + \frac{30{,}240}{1.0(12.0)} = 14{,}650 + 2{,}520 = +17{,}170 \text{ psi}$$

Maximum compressive stress:

$$\frac{-K_o Mc}{I} + \frac{T}{A} = \frac{-0.840(288{,}200)(6)}{1.0(12)^3} + 2{,}520 = -10{,}090 + 2{,}520 = -7{,}570 \text{ psi}$$

* This added distance was made necessary during fabrication of the wye in order to fit the plates together.

Nomograph Method

During the period of this study a means was sought by which the data on similitude of design could be presented. Curves had been made in the past, but these proved cumbersome. Finally it was decided to present the material in the form of a nomograph (Fig. 15). After the authors had investigated the possibility and developed the form, a series of two-plate 90-deg wye branches were designed for pressures from 100 to 600 psi. The resulting crotch depths were found to follow a curved pattern, and this was used to plot the interpolated points on the plate depth scale. Later the pressure scale was extended to 1,200 psi by the same procedure. When the 90-deg wye branch design appeared complete, a series of designs was made at various angles of deflection. These results were compared with the 90-deg designs, and there appeared to be a constant proportion between the two at any particular deflection angle. This proportion, called the N factor, was plotted against the deflection angle of the wye branch (Fig. 16).

Up to this point the authors had considered only sections with equal diameters and two-plate designs. Next a series of one-plate designs was calculated for various ratios of pipe diameters. When the resulting dimensions of these designs were compared with those of the two-plate designs, a relationship was found that depended on the deflection angle. In order to graph the comparisons, it was necessary to plot a family of curves at various deflection angles on a graph of pipe diameter ratio versus a proportion called the Q factor (Fig. 17). These curves fell so close to each other that it seemed simpler to plot only those for 90, 60, and 30 deg and interpolate between them. There remained one other important dimension of the reinforcing plate to consider—that at the top of the sections. When this was investigated, it was found to vary quite closely in a good design with the smaller crotch depth (d_b). By checking through all the past designs, it was possible to plot another set of curves of top depth versus base depth. One of these curves was based on the one-plate designs, the others on the two-plate designs at various deflection angles (Fig. 18).

In examining the equations of similitude and their relation to this work, a method of converting the depth and thickness of plate for curved beams was sought. Using the section modulus equation did not give accurate results in this case, because it did not hold the stresses constant as the plate was varied in cross section. By empirical methods a conversion equation was derived by which the depth and thickness of plate could be varied so that the depth was never more than 30 times the thickness. This equation was based on test results and was checked by being used on sections which were later tested. From this it was assumed to be reasonably accurate.

In order to check the nomograph, complete the curves, and derive the equations used, a great number of examples was required. These were mostly based on theoretical knowledge. Test results had been used throughout as a basis for the design data, but it was still necessary to make other tests to check the curves as they were formed. In this series of complete tests on sections which were based on the nomograph method of design, the stresses in the critical sections were analyzed to ascertain their effect on any curves used in the design. From

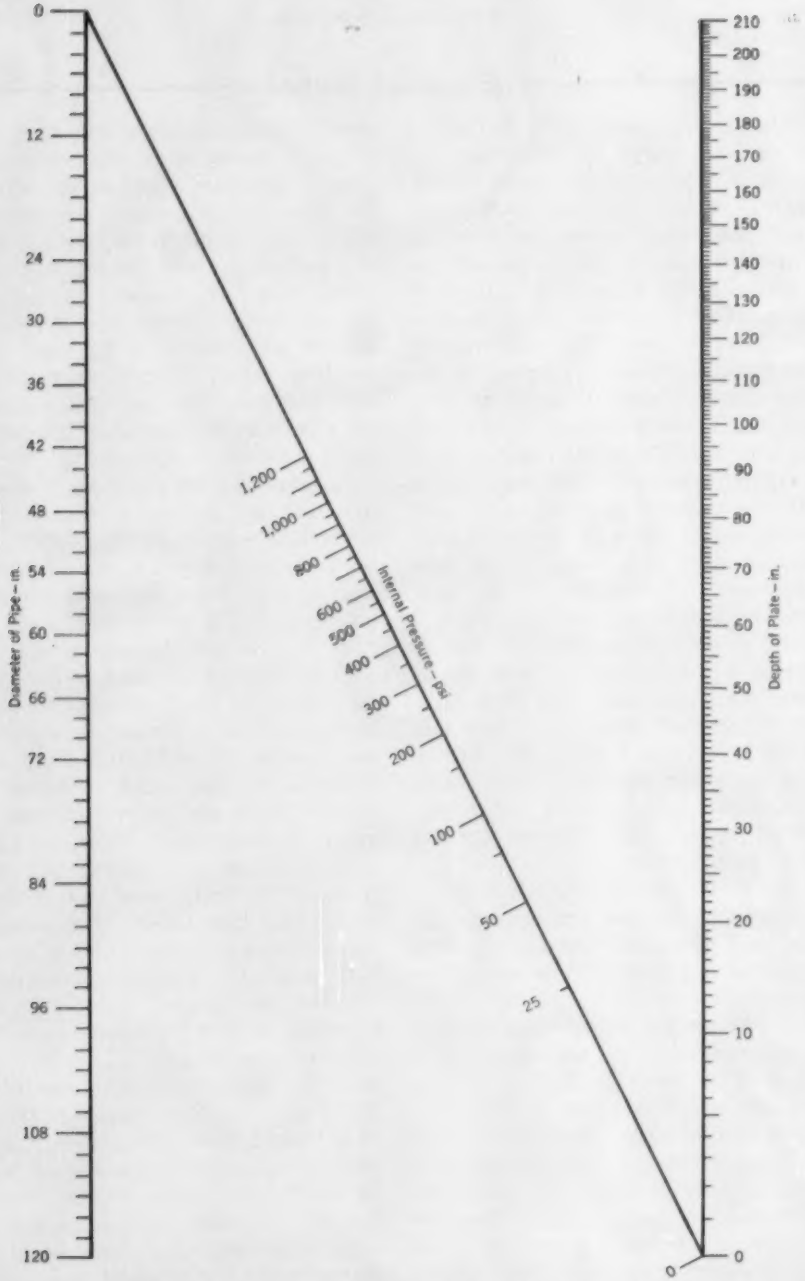


Fig. 15. Nomograph for Selecting Reinforcement Plate Depths of Equal-Diameter Pipes

Plate thickness, 1 in.; deflection angle, 90 deg.

this information the curves were adjusted by making the design pressure that pressure which gave the section a stress of 30,000 psi at its most critical point. An effort was made to balance the stresses in the various parts so that each had a stress close to the critical stress in magnitude.

On the following pages the nomographic method of design is illustrated, along with examples and test results. Superimposed on the drawings of the tested sections are the stresses which occurred on the inner and outer fibers of the reinforcement plates. The tension stresses were plotted outward and the compression stresses inward.

Nomograph Design

Step 1. Lay a straightedge across the nomograph (Fig. 15) through the appropriate points on the pipe diameter (see Step 2b) and internal-pressure scales; read off the depth of plate from its scale. This reading is the crotch depth for 1-in. thick plate for a two-plate 90-deg wye branch pipe.

Step 2a. If the wye branch deflection angle is other than 90 deg, use the N factor curve (Fig. 16) to get the factors which, when multiplied by the depth of plate found in Step 1, will give the wye depth, d_w, and the base depth, d_b, for the new wye branch.

Step 2b. If the wye branch has unequal-diameter pipe, the larger-diameter pipe will have been used in Steps 1 and 2a, and these results should be multiplied by the Q factors found on the single-plate stiffener curves (Fig. 17) to give d'_w and d'_b. These factors vary with the ratio of the radius of the small pipe to the radius of the large pipe.

Step 3. If the wye depth, d_w, found so far is greater than 30 times the thickness of the plate (1 in.), then d_w and d_b should be converted to conform to a greater thickness, t, by use of the general equation:

$$d = d_1 \left(\frac{t_1}{t} \right)^{0.917 - \frac{\Delta}{340}}$$

in which d_1 is the existing depth of plate; t_1 is the existing thickness of plate; d is the new depth of plate; t is the new thickness of plate selected;

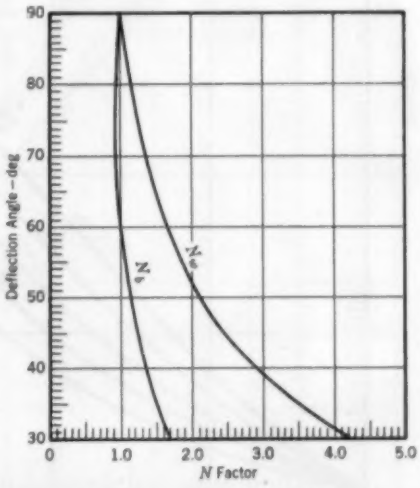


Fig. 16. N Factor Curves

For wyes with deflection angles from 30 to 90 deg, the N *factors obtained from the above curves are applied to the plate depth,* d, *found from the nomograph (Fig. 15), in accordance with the equations:*

$$d_w = N_w d; \; d_b = N_b d.$$

and Δ is the deflection angle of the wye branch.

Step 4. To find the top depth, d_t or d'_t, use Fig. 18, in which d_t *or* d'_t is plotted against d_b or d'_b. This dimension gives the top and bottom depths of plate at 90 deg from the crotch depths.

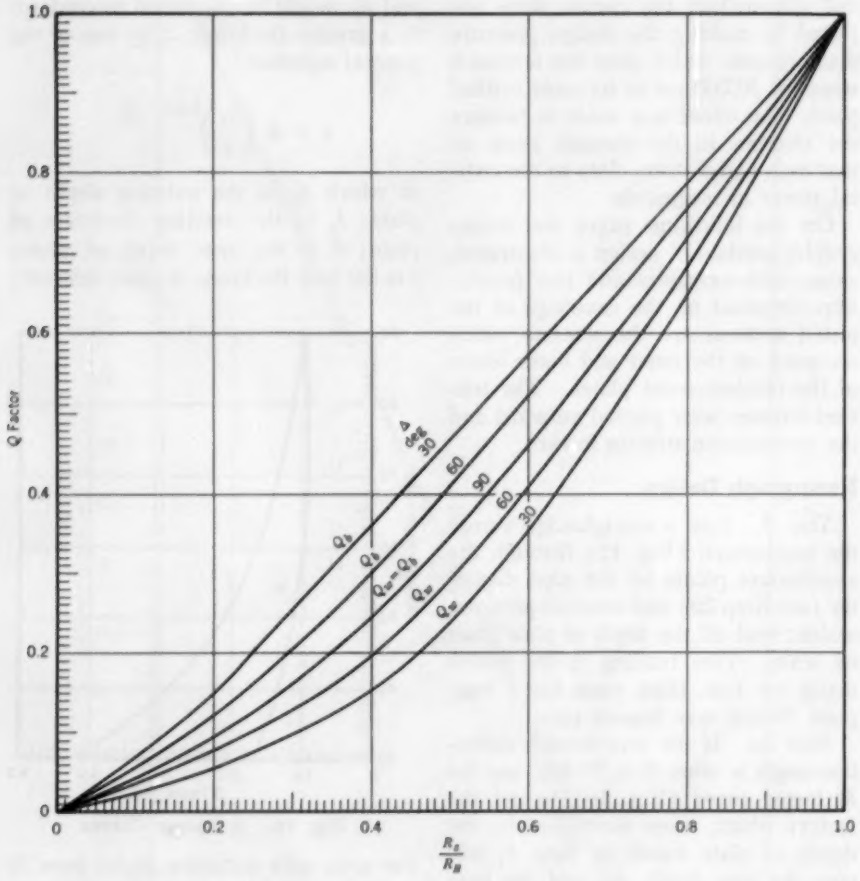


Fig. 17. *Q* Factor Curves

For pipes of unequal diameter, find d_w *and* d_b *for the larger-diameter pipe (from Fig. 15 and 16); then:* $Q_w d_w = d'_w$, *crotch depth of single-plate stiffener; and* $Q_b d_b = d'_b$, *base depth of single-plate stiffener.*

Step 5. The interior curves follow the cut of the pipe, but the outside crotch radius in both crotches should equal d_t plus the radius of the pipe, or, in the single-plate design, d'_t plus the radius of the smaller pipe. Tangents connected between these curves complete the outer shape.

The important depths of the reinforcement plates, d_w, d_b, and d_t (Fig. 19) can be found from the nomograph. If a curved exterior is desired, a radius equal to the inside pipe radius plus d_t can be used, both for the outside curve of the wye section and for the outside curve of the base section.

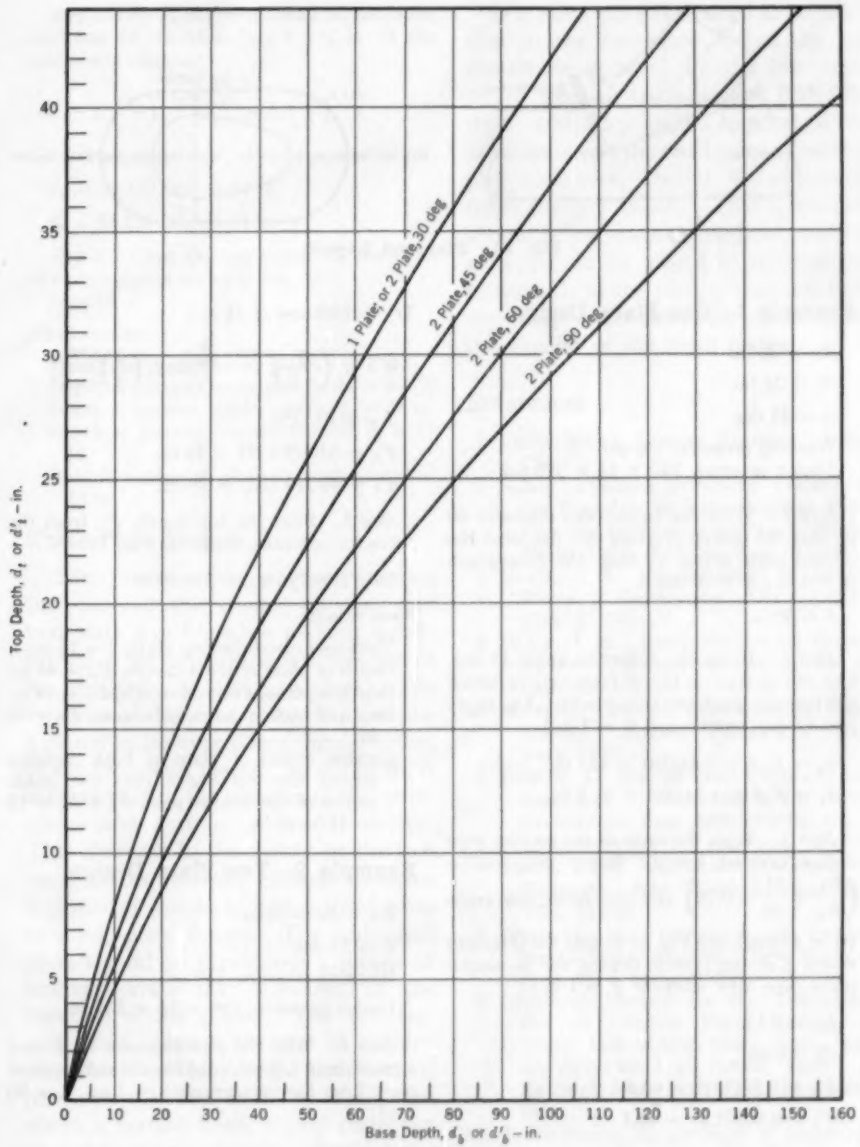


Fig. 18. Selection of Top Depth

d'_t and d'_b are one-plate design dimensions; d_t and d_b are two-plate design dimensions.

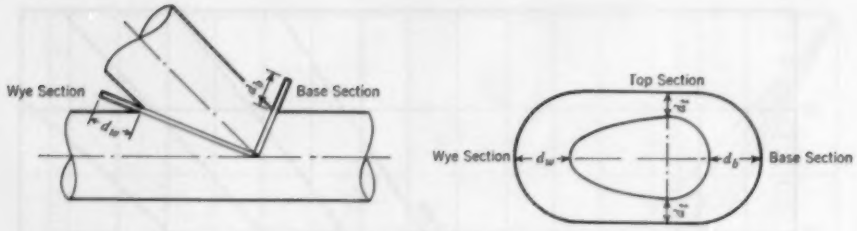


Fig. 19. Plan and Layout

Example 1—One-Plate Design

$R_B = 30$ in.

$R_s = 21$ in.

$\Delta = 45$ deg

Working pressure, 230 psi
Design pressure, $230 \times 1\frac{1}{2} = 350$ psi

Step 1. With the larger pipe diameter 60 in. and the design pressure 350 psi, read the critical plate depth, d, from the nomograph ($t = 1$ in., $\Delta = 90$ deg):

$d = 50$ in.

Step 2. Using the deflection angle 45 deg, find the factors on the N factor curve which will convert the depth found in Step 1 to apply to a 45-deg wye branch ($t = 1$ in.):

$d_w = K_w d = 2.45(50) = 122$ in.

$d_b = K_b d = 1.23(50) = 61.5$ in.

Step 3. With the ratio of the smaller pipe radius divided by the larger pipe radius $\left(\frac{R_s}{R_B} = \frac{21}{30} = 0.70\right)$ and the deflection angle ($\Delta = 45$ deg), use Fig. 17 to find the Q factors which give the crotch depths for a single-plate pipe wye stiffener ($t = 1$ in.):

$Q_w = 0.52$

$Q_b = 0.66$

$d'_w = 0.52(122) = 63.4$ in.

$d'_b = 0.66(61.5) = 40.5$ in.

Step 4. As the depth, d'_w, is greater than 30 times the thickness, t, the conversion equation should be used:

$$d = d_1 \left(\frac{t_1}{t}\right)^{0.917 - \frac{45}{360}}$$

Try a thickness of $1\frac{1}{2}$ in.:

$$d = d_1 \left(\frac{1}{1.5}\right)^{0.917 - \frac{45}{360}} = d_1 \left(\frac{2}{3}\right)^{0.792}$$

$d = d_1(0.725)$

$d'_w = 63.4(0.725) = 46$ in.

$d'_b = 40.5(0.725) = 29$ in.

Step 5. Find the top depth, d'_t, from the curve for one-plate design in Fig. 18:

for $d'_b = 29$ in., $d'_t = 18$ in.

Final results:

Thickness of reinforcing plate, $t = 1\frac{1}{2}$ in.
Depth of plate at acute crotch, $d'_w = 46$ in.
Depth of plate at obtuse crotch, $d'_b = 29$ in.
Depth of plate at top and bottom, $d'_t = 18$ in.
Outside radius of plate at both crotches equals the top depth plus the inside radius of the smaller pipe, $d'_t + R_s = 18 + 21 = 39$ in.

Example 2—Two-Plate Design

$R_B = R_s = 36$ in.

$\Delta = 53$ deg

Working pressure, 150 psi

Design pressure, $150 \times 1\frac{1}{2} = 225$ psi

Step 1. With the pipe diameter 72 in. and a pressure of 225 psi, read the critical depth of plate from the nomograph ($t = 1$ in., $\Delta = 90$ deg):

$d = 49$ in.

Step 2. From the N factor curve, find the two factors at $\Delta = 53$ deg; then, at $t = 1$ in.:

$d_w = 1.97(49) = 96.5$ in.

$d_b = 1.09(49) = 53.4$ in.

Step 3. As d_w is greater than 30 times the thickness of the plate, try $t = 2$ in. in the conversion equation:

$$d = d_1\left(\frac{t_1}{t}\right)^{0.917-\frac{\Delta}{360}} = d_1\left(\frac{1}{2}\right)^{0.770}$$

$$= d_1(0.586)$$

$$d_w = 96.5(0.586) = 57 \text{ in.}$$

$$d_b = 53.4(0.586) = 31 \text{ in.}$$

Step 4. Read the top depth, d_t, from the two-plate design curve in Fig. 18:

$d_t = 15$

Final results:

Thickness of reinforcing plate, $t = 2$ in.
Depth of plate at acute crotch, $d_w = 57$ in.
Depth of plate at obtuse crotch, $d_b = 31$ in.
Depth of plate at top and bottom, $d_t = 15$ in.
Outside radius of plate at both crotches, 51 in.

Three-Plate Design

The preceding nomograph section has covered the design of one- and two-plate wye branches without touching on a three-plate design because of its similarity to the two-plate. The function of the third plate is to act like a clamp in holding down the deflection of the two main plates. In doing so, it accepts part of the stresses of the other plates and permits a smaller design. This decrease in the depths of the two main plates is small enough to make it practical simply to add a third plate to a two-plate design. The additional plate should be considered a means of holding down the deflection at the junction of the plates. The two factors which dictate the use of a third plate are diameter of pipe and internal pressure. When both of these are above a certain limit, a ring plate can be used advantageously. During the work on this report, it was concluded that these limits could be 60-in. ID and 300 psi. If either of these factors is below the limit, the designer should be allowed to choose a third plate.

If a third plate is desired as an addition to the two-plate design, its size should be dictated by the top depth (d_t). Because the other two plates are flush with the inside surface of the pipe, however, the shell plate thickness, plus clearance, should be subtracted from the top depth. This dimension should be constant throughout, and the plate should be placed at right angles to the axis of the pipe, giving it a half-ring shape. Its thickness should equal the smaller of the main plates.

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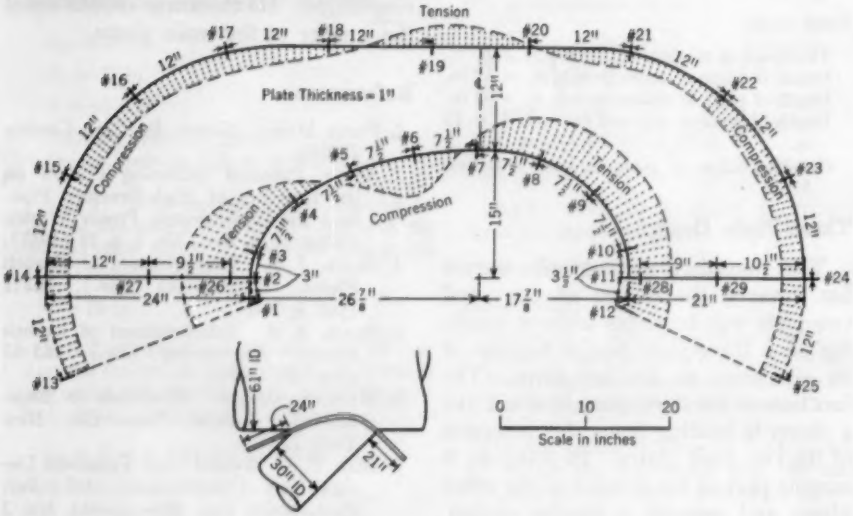
Test Results

One-Plate Wye

A single curved plate serves as reinforcement for this 30 × 61-in., 45-deg wye section (see Fig. 20).



Item	Measured Stress*—lb	Item	Measured Stress*—lb	Item	Measured Stress*—lb
#1	32,000	#11	22,000	#21	−8,000
#2	34,000	#12	23,000	#22	−9,000
#3	31,000	#13	−12,000	#23	−12,000
#4	12,000	#14	−12,000	#24	−12,000
#5	−8,000	#15	−11,000	#25	−12,000
#6	−20,000	#16	−10,000	#26	28,000
#7	7,000	#17	−11,000	#27	0
#8	21,000	#18	−4,000	#28	22,000
#9	25,000	#19	10,000	#29	3,000
#10	23,000	#20	1,000		

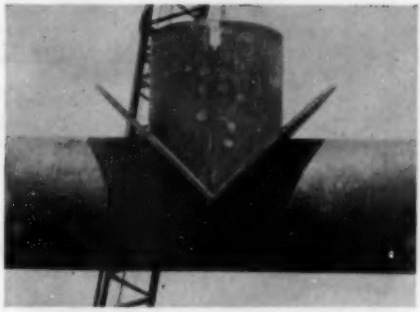
* At 250-psi pressure.

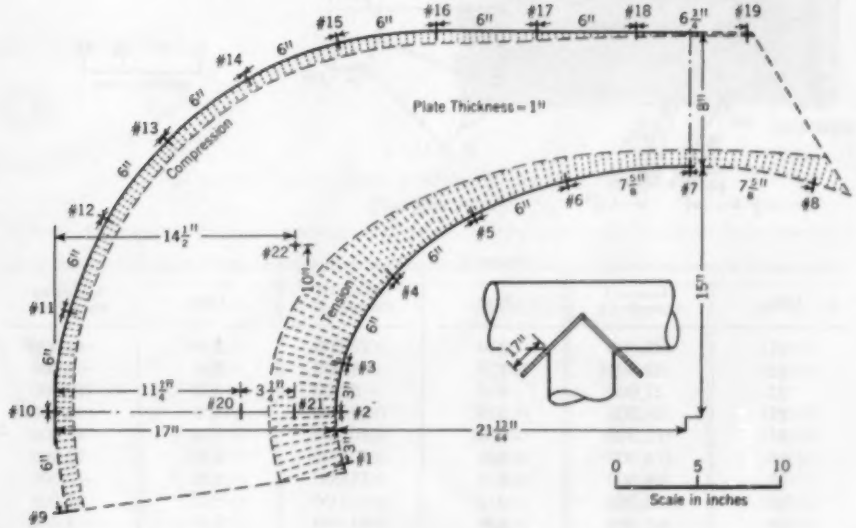
Fig. 20. One-Plate Design—Plan of Stiffener Plate

Data: $R_B = 30.5$ *in.;* $R_s = 15$ *in.;* $\Delta = 45$ *deg;* $p = 250$ *psi. See italic note below Fig. 7.*

Two-Plate Wye

This 30 × 30-in., 90-deg wye section has two crotch plates (see Fig. 21).



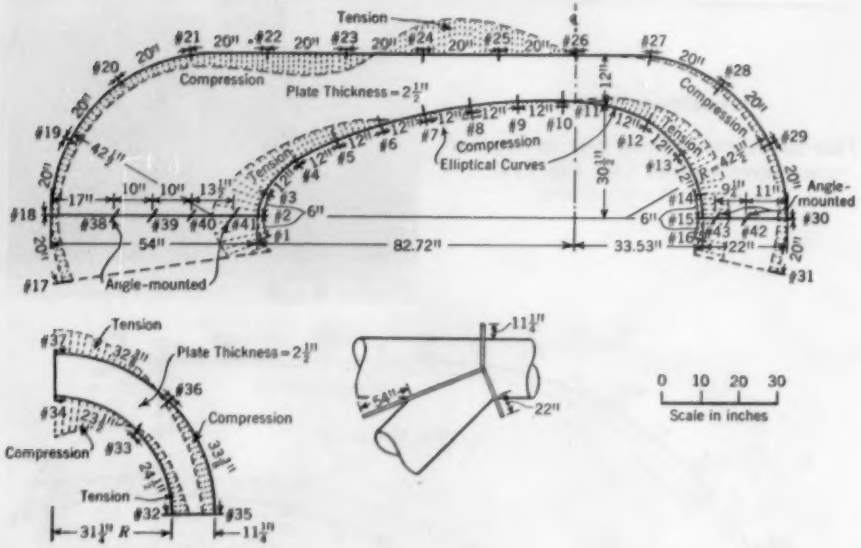


Item	Measured Stress*—lb	Item	Measured Stress*—lb	Item	Measured Stress*—lb
#1	30,000	#9	−7,000	#16	−5,000
#2	30,000	#10	−8,000	#17	−4,000
#3	30,000	#11	−7,000	#18	−1,000
#4	24,000	#12	−7,000	#19	−1,000
#5	15,000	#13	−7,000	#20†	6,000
#6	11,000	#14	−6,000	#21†	15,000
#7	7,000	#15	−6,000	#22†	6,000
#8	14,000				

* At 360-psi pressure. † Principal stresses of rosettes.

Fig. 21. Two-Plate Design—Plan of Stiffener Plates

Data: $R_B = 15$ in.; $R_s = 15$ in.; $\Delta = 90$ deg; $p = 360$ psi. See italic note below Fig 7. See also Table 8 (page 627).



Item	Measured Stress*—lb	Item	Measured Stress*—lb	Item	Measured Stress*—lb
#1	25,000	#16	22,000	#30	−6,000
#2	34,000†	#17	−8,000	#31	−7,000
#3	27,000	#18	−6,000	#32	16,000
#4	16,000	#19	−9,000	#33	3,000
#5	12,000	#20	−10,000	#34	−31,000
#6	4,000	#21	−10,000	#35	−10,000
#7	−4,000	#22	−15,000	#36	−2,000
#8	−8,000	#23	−13,000	#37	17,000
#9	−5,000	#24	23,000	#38	−3,000
#10	−1,000	#25	24,000	#39	−1,000
#11	5,000	#26	0	#40	4,000
#12	13,000	#27	−2,000	#41	28,000
#13	21,000	#28	−5,000	#42	1,000
#14	21,000	#29	−6,000	#43	11,000
#15	21,000				

* At 335-psi pressure. † 22,000 at 200 psi.

Fig. 22. Three-Plate Design—Plan of Stiffener Plates

Data: $R_B = 30.5$ *in.;* $R_s = 30.5$ *in.;* $\Delta = 45$ *deg;* $p = 300$ *psi. See italic note below Fig. 7.*

Three-Plate Wye

In this 61 × 61-in., 45-deg wye, the plate on the back is not welded to the pipe shell (see Fig. 22).

TABLE 8

Measured Stresses for 30 × 30-in., 90-deg Two-Plate Wye (See Fig. 21)

Gage	Stress—*lb**				
	100 psi	200 psi	300 psi	360 psi	400 psi
#1	9,400	18,500	26,500	30,400	33,000
#2	8,600	17,300	25,400	29,500	32,300
#3†					
#4	5,600	13,100	20,000	24,200	27,000
#5	3,300	7,500	12,300	15,200	17,100
#6	2,500	5,600	9,400	11,300	12,500
#7	1,700	3,500	5,400	7,100	8,300
#8	2,900	7,100	12,100	14,400	16,000
#9	−1,700	−3,800	−5,600	−7,100	−8,100
#10	−1,700	−4,100	−6,000	−7,600	−8,600
#11	−1,500	−3,600	−6,000	−7,400	−8,300
#12	−1,700	−3,600	−5,400	−7,000	−8,000
#13	−1,800	−3,800	−5,400	−6,800	−7,800
#14	−1,500	−3,600	−5,000	−6,300	−7,100
#15	−1,200	−3,300	−4,700	−5,800	−6,600
#16	−1,200	−3,300	−4,500	−5,400	−6,000
#17	− 800	−2,300	−2,900	−3,600	−4,100
#18	− 200	− 900	−1,100	−1,500	−1,700
#19	− 200	− 800	− 800	−1,200	−1,400
#20‡	2,500	3,100	4,900	5,700	6,200
#21‡	4,200	7,800	12,300	14,800	16,500
#22‡	1,600	3,000	4,900	5,600	6,100

* Minus sign indicates compression.
† Gage No. 3 failed but was considered to have the same stresses as No. 1 because the two were symmetrically placed.
‡ Rosettes.

APPENDIX A

Comparison of Sulzer and Nomograph Design

In 1941 Sulzer Brothers reported (*2* *) a wye branch which withstood an internal pressure of 2,410 psi before it failed. For the sake of comparison, their data were converted from the metric system into inches and pounds, and the present authors' nomograph method of design was used to obtain values for overall length and width of the reinforcing plate. The results are compared in Table 9.

TABLE 9

Comparison of Designs

Item	Sulzer	Nomograph
Main pipe diameter—*in.*	23.6	23.6
Branch pipe diameter—*in.*	9.8	9.8
Deflection angle—*deg*	60	60
Service pressure—*psi*	750	
Test pressure—*psi*	1,140	1,140
Failure pressure—*psi*	2,410	
Shell stress at failure—*psi*	64,000	
Thickness of reinforcing plate—*in.*	0.79	0.79
Overall length of collar—*in.*	31.5	35.2
Overall width of collar—*in.*	26.3	27.4

APPENDIX B

Deflection Nomograph

The nomograph in Fig. 23, based on test data, can be used for finding deflections that will occur at the junction of the reinforcement plates in two- and three-plate wye branches. In the example represented by the broken line—a 54-in. ID welded-steel pipe at a design pressure of 350 psi—the deflection for a three-plate design is found to be 0.068 in.; for a two-plate, 0.098 in.

APPENDIX C

Results of Overstress

During the testing of the various wye branch pipes in this report, there was opportunity to observe their actions when overstress occurs. In the later tests, when the designs were cut to a minimum and strain gages were placed inside the sections, the critical areas of stress reached the yield point very close to the test pressure. When additional pressure was applied to the section, the strain gages on the inside at the crotch appeared to act in an erratic manner. When the internal pressure was released, a number of gages failed to return to their zero reading. Examination of these gages showed that the steel in their vicinity was permanently deformed. In still other areas, the steel was under compression. It appeared that, under high stresses, the section acted in a plastic manner and distributed stresses from critical areas to less critical ones by yielding.

* See reference list, page 623.

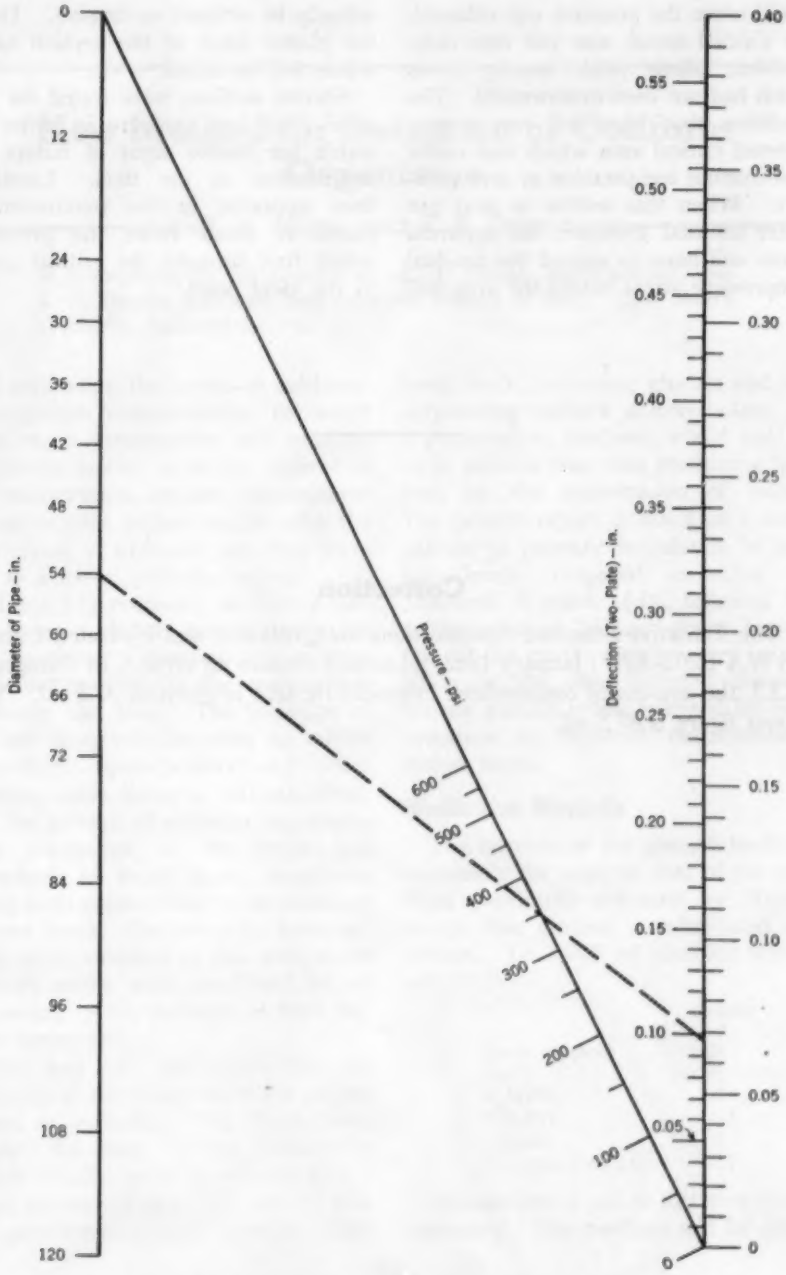


Fig. 23. Nomograph for Deflections in 45-deg Wyes

Then, when the pressure was released, the yielded metal was put into compression, along with nearby areas which had not been overstressed. The condition that remained was a prestressed critical area which was under a permanent compression at zero pressure. When this section is next put under internal pressure, the apparent stress will have to exceed the residual compressive stress before the area will actually be stressed in tension. Thus, the elastic limit of the section as a whole will be raised.

Several sections were tested far beyond their test pressure in order to watch for visible signs of failure or deformation in the steel. Lueder's lines appeared in the reinforcement plates at about twice the pressure which first brought the critical areas to the yield point.

Correction

The Tentative Standard Specifications for Quicklime and Hydrated Lime—AWWA B202–52T (January 1953 JOURNAL) contain an error. In Paragraph 4C.2.3, the amount of concentrated hydrochloric acid is given as 37.5 ml. The correct figure is 87.5 ml.

New Method for Detection of Coliform Organisms

A. A. Hajna and S. R. Damon

A contribution to the Journal by A. A. Hajna, Bacteriologist, and S. R. Damon, Director, both from the Bureau of Labs., State Board of Health, Indianapolis, Ind.

IN estimating the extent of coliform-organism contamination of water supplies, a presumptive test medium containing lactose is usually utilized in the examination, because the coliform group ferment lactose rapidly with the production of acid and gas, thus serving as a useful pollution index.

Hajna (*1*) proposed, in 1951, a buffered desoxycholate lactose broth for use as either a presumptive or confirmatory medium in the examination of water and food. The inclusion of sodium desoxycholate was to inhibit growth of sporeformers and other Gram-positive bacteria without affecting the growth of coliform organisms. Gas production in the broth was abundant in most cases, sometimes being even greater than in the standard lactose broth. Occasionally, however, only small amounts of gas were noted in both media when incubated for 48 hr, owing to the presence of slow lactose fermenters.

McCrady (*2*) has stated that the majority of laboratory workers are opposed to excluding from the coliform group the slow lactose fermenters, which usually ferment glucose with a large amount of gas. In view of this, the question arose as to whether a buffered broth containing glucose and incorporating sodium desoxycholate, as a presumptive medium, would not be more suitable than that containing lactose for the examination of water. The present report is based on a comparison of primary inoculation in lactose broth, prepared according to *Standard Methods* (*3*), followed by confirmation in brilliant-green lactose bile (2 per cent), with primary inoculation in the new buffered desoxycholate glucose broth, followed by confirmation in buffered desoxycholate lactose broth.

Media and Methods

The formula of the glucose broth is essentially the same as that of the medium previously reported by Hajna, except that glucose is substituted for lactose. To 1,000 ml distilled water, add:

	Quantity g
Bacto tryptose	20
NaCl	5
K_2HPO_4	4
KH_2PO_4	1.5
Glucose	5
Na desoxycholate	0.1

Adjustment of pH or filtration is not necessary. The medium will be clear,

having a final pH of 6.9–7.0 after autoclaving at 117°C for 15 min. For 10-ml portions of inoculum, double the amount of ingredients per liter and dispense 10-ml quantities in 20-mm by 150-mm tubes which have inner tubes. For inoculations of 1 ml or less, single strength medium is dispensed in 10-ml amounts in 16 × 150-mm tubes which have inner tubes.

Using *Standard Methods* lactose broth as the presumptive test medium and *Standard Methods* brilliant-green lactose bile (2 per cent) as the confirmatory medium, the efficiency of the new procedure was determined in detecting members of the coliform group. Buffered desoxycholate glucose broth was used as the presumptive test medium and buffered desoxycholate lactose broth as the confirmatory medium.

The choice of buffered desoxycholate lactose broth for confirmation of coliform bacteria from the desoxycholate glucose broth was based on the greater value of desoxycholate lactose as a confirmatory test medium over the brilliant-green lactose bile (2 per cent). Hajna (*1*), comparing the efficiency of the secondary-tube confirmation for the

TABLE 1

*Comparative Effectiveness of BDG * and SM † in Recovery of Coliform and Other Bacteria From Water*

Number of Samples	Sample	Primary Medium	Number of Tubes Inoculated	Number of Tubes Showing Gas	Number of Tubes Confirmed for Coliform Organisms	Other Bacteria Recovered		Bacteria Identified in Absence of Gas in Primary Medium	
						Type	Number of Tubes	Type	Number of Tubes
129	Untreated water	BDG	2,580	917	901	*P.‡ morgani*	4	No growth	50
						P.‡ vulgaris	3	*Ps.§ aeruginosa*	54
						P.‡ mirabilis	3	*Chr.# violaceum*	12
						Ps.§ aeruginosa	5	*S.** marcescens*	11
						Esch. coli‖	1	*Esch. coli‖*	59
								Aer. aerogenes	5
								P.‡ veligeri	17
		SM	2,580	1,103	866	No growth	231	No growth	75
						Ps.§ aeruginosa	6	*Ps.§ aeruginosa*	39
						Bethesda paracolon		*Chr.# violaceum*	18
								Ser.†† marcescens	5
								Esch. coli‖	51
								P.‡ veligeri	2
								Aer. aerogenes	1
550	Treated water	BDG	11,000	82	62	*P.‡ morgani*	16	*Chr.# violaceum*	3
						P.‡ mirabilis	2	*S.** marcescens*	4
						P.‡ vulgaris	1	*Esch. coli‖*	6
						Bethesda paracolon	1	*Ps.§ aeruginosa*	3
								Bacteroides	5
		SM	11,000	97	59	*Esch. coli‖*	7	*Esch. coli‖*	4
						No growth	31	*Ps.§ aeruginosa*	2
Total		BDG	13,580	999	963				
		SM	13,580	1,199	925				

* Buffered desoxycholate glucose broth.
‡ *Proteus.*
‖ Anaerogenic.
** *Salmonella.*
† *Standard Methods* lactose broth.
§ *Pseudomonas.*
# *Chromobacterium.*
†† *Serratia.*

coliform group from standard lactose broth showing gas in 7,669 tests, obtained 7,499 confirmations of the coliform group with desoxycholate lactose broth, against the 7,263 with the brilliant-green lactose bile (2 per cent). Where there was growth in the desoxycholate glucose broth with no visible gas formation, plating media (Bacto MacConkey agar * and BBL † desoxycholate lactose agar) were employed, with colonies picked to differential test media for identification (*4*). Similarly, where gas was produced in the primary media, but not in the secondary, plating media were used to recover other types of Gram-negative bacteria where possible.

Shucked oysters were examined for the coliform group by the recommended method of procedure (*5*).

The bacteriological examination of crabmeat and pecans was carried out as advised by Hunter and Ostrolenk, US Food and Drug Administration.

Discussion of Results

Results obtained in comparing the value of the new method—primary inoculation of buffered desoxycholate glucose broth followed by confirmation for coliform organisms in buffered desoxycholate lactose broth—against that of the *Standard Methods* media, are given in Tables 1, 2, 3. Different types of water samples—a total of 679 of which 129 were untreated and 550 were treated—were examined for coliform content, as well as for other types of bacteria. The total number of food samples examined was 292, of which 80 were shucked oysters, 120 shelled pecans, and 92 crabmeat. The choice was made because these items are usually processed by human hands, and are not worked on mechanically.

In the primary inoculations it was noted that 3,352 tubes of the buffered desoxycholate glucose broth were positive, as against 3,418 tubes of standard lactose broth. Carrying these tubes on to confirmation in buffered desoxycholate lactose, it was found that 3,257 or 97.2 per cent of those positive in the buffered desoxycholate glucose broth were confirmed, whereas 3,126 or 91.5 per cent of the 3,418 which showed gas were confirmed in the *Standard Methods* lactose broth.

Prescott, Winslow, and McCrady (*6*) have stated that other organisms contained in the sample may reduce the amount of gas produced in standard lactose broth by typical coliform organisms and that the volume of gas produced cannot be accepted as a sure indication of the type of organism present. This has been confirmed in the authors' work, along with the isolation of various types of bacteria other than the coliform group. For example, some of the paracolons of the Bethesda-Ballerup group which have been isolated, in Edwards' (*7*) scheme, ferment lactose with gas production. In the present investigation, rapid lactose-fermenting Bethesda-Ballerup group paracolons have been labeled as coliform bacteria.

In the examination of food, the total number of confirmations and the percentage of tubes confirmed are negligible, except for the pecans.

Seventeen slow lactose-fermenting coliforms were isolated. These produced very little gas in standard lactose broth and secondary confirmatory

* A product of Difco Labs., Inc., Detroit.
† Baltimore Biological Lab., Baltimore.

broth. The same cultures, when transplanted to desoxycholate glucose broth, produced abundant gas. When they were transferred from the new broth to the secondary tubes of desoxycholate lactose broth, abundant gas was also produced, but not as much as from the glucose broth. This may possibly be explained by a rejuvenation of the slow lactose-fermenting organisms.

Isolation of other types of Gram-negative bacteria does not seem desirable. The only exception may be the isolation and confirmation of such bacteria from treated water. Although pathogenic bacteria of the typhoid-paratyphoid-dysentery group have not been isolated, others, such as the *Proteus* group (see Table 1), were recovered. Their pathogenicity may be questionable, yet they cannot be ignored, for they may cause obscure gastrointestinal disturbances in children and older people.

TABLE 2

*Comparative Effectiveness of BDG * and SM † in Recovery of Coliform and Other Bacteria From Food*

Number of Samples	Sample	Primary Medium	Number of Tubes Inoculated	Number of Tubes Showing Gas	Number of Tubes Confirmed for Coliform Organisms	Other Bacteria Recovered		Bacteria Identified in Absence of Gas in Primary Medium	
						Type	Number of Tubes	Type	Number of Tubes
80	Shucked oysters	BDG	1,200	752	752	None	0	No growth *P.‡ mirabilis*	16 40
		SM	1,200	753	753	None	0	No growth *P.‡ mirabilis*	12 29
120	Shelled pecans	BDG	1,800	621	562	No growth *P.‡ morgani* *P.‡ vulgaris* *P.‡ mirabilis* Bethesda paracolon	6 9 6 21 17	*Ps.§ aeruginosa* *Esch. coli*‖	8 8
		SM	1,800	489	472	Slow lactose fermenters	17	No growth *Esch. coli*‖	97 5
92	Crabmeat	BDG	1,380	980	980	None	0	No growth *Ps.§ aeruginosa* *P.‡ vulgaris* *P.† mirabilis* Bethesda paracolon	328 44 3 3 6
		SM	1,380	976	976	None	0	No growth *Ps.§ aeruginosa* *P.‡ vulgaris* *P.‡ mirabilis* *P.‡ morgani* Bethesda paracolons	339 39 2 5 11 8
Total		BDG	4,380	2,353	2,294				
		SM	4,380	2,218	2,201				

* Buffered desoxycholate glucose broth.
† *Standard Methods* lactose broth.
‡ *Proteus.*
§ *Pseudomonas.*
‖ Anaerogenic.

Summary

1. The formula has been presented for a new presumptive medium—buffered desoxycholate glucose broth—to replace standard lactose broth in the primary inoculation of samples of water and food.

2. A new procedure has been described for determining the density of coliform bacteria in samples, by transfer from buffered desoxycholate glucose broth to a confirmatory medium, buffered desoxycholate lactose broth, in place of the *Standard Methods* media.

3. More positive confirmed findings result from the employment of the new method than from the *Standard Methods* procedure.

EDITOR'S NOTE: Comparisons on the basis of total confirmed tubes tend to mislead investigators because of the masking effect introduced by this method of data presentation. The investigator expects difference in coliform response in a given medium according to the size of the test portion of inoculum, particularly from bacterial crowding, antibiotic effects, and toxic compounds from industrial wastes in the case of some untreated waters. These effects are more often noted in larger than in smaller portions which become highly diluted by the culture medium or otherwise. It seems that this difficulty is reduced when a given combination of confirmed tube results is expressed by its corresponding MPN rather than by accumulating the influence on a total tube basis. Properly analyzed, the MPN's might show more striking contrasts not indicated when total tube comparisons are made. Apparent agreements might then become doubtful or turn into disagreements.

TABLE 3

*Comparative Effectiveness of BDG * and SM † in Recovery of Coliform Bacteria From Water and Food*

Number of Samples	Sample	Primary Medium	Number of Tubes Inoculated	Number of Tubes Showing Gas	Number of Tubes Confirmed for Coliforms	Per Cent of Tubes Confirmed for Coliforms
679	Water	BDG	13,580	999	963	96.4
		SM	13,580	1,199	925	77.1
292	Food	BDG	4,380	2,353	2,294	97.5
		SM	4,380	2,218	2,201	99.2
971 Total	Water and food	BDG	17,960	3,352	3,357	97.2
		SM	17,960	3,418	3,126	91.5

* Buffered desoxycholate glucose broth.
† *Standard Methods* lactose broth.

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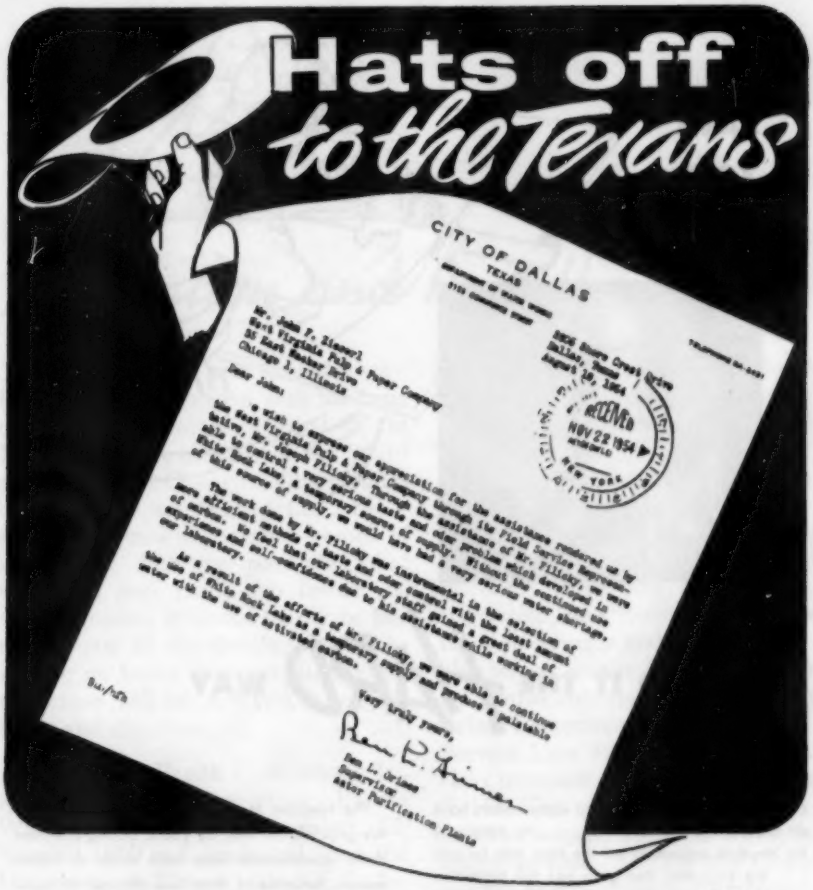

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The Neptune Meter Company has adhered to this principle for over 50 years. During that time many improvements have been made to Trident meters. Yet none of these has affected the basic design of the meters. The oldest Trident can be modernized with new, improved Trident parts. Rebuilt or brand-new, Trident is still the finest meter you can buy. Ask your neighbor . . . or ask the Neptune representative next time he calls.

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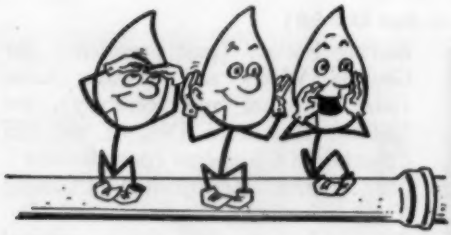

Percolation and Runoff

AWWA's 75th year is under way, having officially rung in with the bang of President Amsbary's gavel at the Friday, Jun. 17, meeting of the Board of Directors in Chicago. Other things happened in the Windy City, too—such as the new registration record of 2,313—but, with the possibility that this issue may reach you before the meeting opens, it would hardly be fair to give you all the details. It will be spilling no beans, however, to tell you that these will be AWWA's new officers and directors:

President—Frank C. Amsbary Jr., vice-president and secretary, Illinois Water Service Co., Champaign, Ill. Born in 1900 and graduated from the University of Illinois with a C.E. degree in 1922, he is a registered professional engineer in Illinois. He joined the Champaign-Urbana Water Co., predecessor of his present firm, in 1923 as supervising engineer. He became manager in 1927, director in 1936, secretary in 1937, and vice-president in 1939. He continues to hold these posts, and in addition is vice-president of the Northern Illinois Water Corp.

Amsbary joined AWWA in 1927, and has distinguished himself in a host of activities. He has been trustee, vice-chairman, chairman, and director of the Illinois Section, which gave him its Fuller Award in 1941. He has held the posts of secretary-treasurer, vice-chairman, and chairman of the AWWA Plant Management and Operation Division and secretary of its successor, the Water Works Management Division. Committees of which he has been chairman include those on Water Use in Air Conditioning and Other Refrigeration, and Recommended Design for Valve Boxes and Covers. He has also served on the Committee on Water Works Administration, the Goodell Prize Committee, the Committee on Service Line Materials, and the Special Committee of 1941 on the Nicholas S. Hill Cup.

Other organizations in which he is active include ASCE; the National Society of Professional Engineers; the Public Utility Advisory Committee of the College of Commerce, University of Illinois; and two college engineering societies: Theta Tau and Musan.

Vice-President—Paul Weir, general manager, Atlanta Water Works. Born in 1906, he holds a degree in civil engineering from the Georgia Institute of Technology (1928) and is a registered professional engineer in Georgia. His quarter century of service with the Atlanta water department includes 11 years as superintendent of the purification plant and 5 years as assistant

(Continued on page 36 P&R)

(Continued from page 35 P&R)

V.P.—Weir

Treasurer—Brush

general manager and engineer; he was named to his present post in 1947.

An AWWA member since 1929, he has been vice-chairman (1938), chairman (1939), and director (1943–46) of the Southeastern Section, which tendered him the Fuller Award in 1940. He was chairman of the Fuller Award Society in 1948 and is also a recipient of the Goodell Prize (1941). His activities within the Association have been unusually extensive: He served successively as trustee (1938–39), vice-chairman (1940), and chairman (1941) of the Water Purification Division and as secretary-treasurer (1949), vice-chairman (1950), and chairman (1951) of the Water Works Management Division, as well as having been a Water Resources Division trustee in 1948. His committee memberships have included Public Relations, Water Rates, Compensation of Water Works Personnel, Watershed Protection and Erosion Control, State Highway Legislation, Publication Review, Standards for Threads for Underground Service Line Fittings, National Committee for Utilities Radio, and Special Committee on Field Studies of Performance of Sulfur Jointing Compounds.

Other organizations in which he has been active are ASCE (past president, Georgia Section), the Georgia Engineering Society (past president), the Georgia Water and Sewage Assn. (past president and secretary), the Natural Resources Div. of the US Chamber of Commerce (past director), the Atlanta Metropolitan Planning Commission, the Federal Civil Defense Administration (as a regional member of the Engineering Dept.), and the scholastic engineering fraternity, Chi Epsilon.

Treasurer—William W. Brush, editor of *Water Works Engineering.* Brush was born in Orange, N.J., in 1874 and was educated at New York University, from which he received B.S., C.E., and M.S. degrees. He served as engineer with the Brooklyn Water Dept. from 1894 to 1907, transferring to the New York Board of Water Supply in the latter year. In 1910, he was shifted to the city's operating division, the Dept. of Water Supply, Gas, and Electricity, as deputy chief engineer of water supply. In 1927 he was appointed chief engineer of water supply, and served in that capacity until 1934, when he retired after a cumulative total with the three organizations of 40 years of service. He then accepted his present post as editor of *Water Works Engineering.*

His reelection as treasurer continues an unbroken record of 33 years of high office in the Association, for, except for a 2-year interval in 1928–29 when he was successively vice-president and president, Brush has been holding AWWA's purse-strings since 1922. In that time also he has been active on many committees, and has served ex officio as a member of both the Board of Directors and its Executive Committee. He received the John M. Diven Medal in 1932 and in 1937 was made an Honorary Member.

(Continued on page 38 P&R)

EASY DOSE IT...

...at the Bland Correctional Farm

Operating personnel at the Bland Correctional Farm in White Gate, Virginia, report this Builders Model DVS Chlorinizer has been mighty easy to work with ever since it was installed in early 1951. Due to Chlorinizer's positive-acting chlorine control system, plant personnel have been able to set the chlorine dosage at the exact rate required by their water sterilization needs.

Plant operator Johnson is shown in the photo adjusting the feed rate and double checking the adjustment on the direct-reading Sightflo indicator which meters total chlorine flow. Beyond the replacement of chlorine cylinders and an occasional check on chlorine flow rate, the Builders Chlorinizer has required no appreciable service or maintenance since installation.

Hundreds of performance reports like this are coming in from Chlorinizer users in municipalities, institutions, and industries throughout the country. If you are considering new chlorination capacity, or replacement of present chlorination equipment, why not check your nearest Chlorinizer installation. We will be glad to send you a list of Chlorinizer users in your area . . . and also bulletins describing the four basic Chlorinizer models. Write, today, to Builders-Providence, Inc., 365 Harris Avenue, Providence 1, Rhode Island.

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BIF METERS FEEDERS CONTROLS

(Continued from page 36 P&R)

Canadian—
Armstrong

Indiana—
Williams

SECTION DIRECTORS

Canadian—C. G. Russell Armstrong, consulting engineer, Windsor, Ont. Born at Merlin, Ont., he was graduated from the University of Toronto with a B.A.Sc. degree and later obtained an Ontario land surveyor degree. His early engineering experience was with the International Nickel Co. at Copper Cliff, Ont. He taught at the University of Toronto for 2 years and in 1923 formed a partnership with the late J. J. Newman, as consulting engineers and land surveyors. In 1944 Mr. Armstrong became sole owner of the business, which is now in his name. He has a very large municipal practice.

An AWWA member since 1934, he is a past chairman of the Canadian Section. Other organizations with which he is affiliated include the Ontario Land Surveyors Assn. (past president), the Engineering Institute of Canada (past vice-president), and the Association of Professional Engineers of the Province of Ontario.

Indiana—Clyde E. Williams, senior partner, Clyde E. Williams and Assocs., South Bend. Born in 1902, he holds a degree in civil engineering from the University of Michigan. He served as sanitary engineer for the Indiana Board of Health from 1933 until he became city civil engineer at South Bend in 1939, a position he held until 1947. The civil and sanitary engineering consultant firm that bears his name was formed in 1941 and was changed to a partnership in 1952.

He has been an AWWA member since 1934 and was chairman of the Indiana Section in 1948–49. Other organizations to which he belongs include ASCE, FSIWA, and the National Society of Professional Engineers.

Iowa—Clifford W. Hamblin, superintendent of public works, Mason City. Born at Mason City in 1909, he holds a degree in civil engineering from Iowa State College (1934). In 1934–37 he worked as a junior engineer for the US Dept. of Interior; in 1937–42, as a junior civil engineer for the US Army Corps of Engineers; in 1942–43, as a ground control engineer for Aero Service Corp., Philadelphia; and in 1943–45, as a construction engineer for E. I. du Pont de Nemours & Co. He has been employed by Mason City in various posts since 1945.

He joined AWWA in 1947 and was Iowa Section chairman in 1952–53. Other organizations to which he belongs include ASCE, FSIWA, the American Public Works Assn., the Iowa Engineering Society, and the North Iowa Technical Assn.

Iowa—Hamblin

Kansas—Badley

(Continued on page 40 P&R)

22 MINUTES
for Complete Hydrant Inspection
WITH RENSSELAER

The illustrations show how easy it is to remove and inspect the entire mechanism of the Rensselaer-Corey Fire Hydrant which has been the standard in many cities for more than half a century.

The parts can be removed, inspected, and replaced in twenty-two minutes. No digging for repairs or for standpipe breakage. This hydrant opens with the pressure, by means of the powerful knuckle joint. Easy to operate and cannot stick. It is quick opening with free flow long radius curves. Hydrant will not flood.

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SALES REPRESENTATIVES IN PRINCIPAL CITIES

Division of The Ludlow Valve Manufacturing Co., Inc.

(Continued from page 38 P&R)

Missouri—Hartung

N.C.—Franklin

Kansas—Harry W. Badley, sales representative, Neptune Meter Co., Salina. Born at Boone, Iowa, in 1896, he attended Iowa State College and is a registered professional engineer in Iowa. After his service in World War I, he was engaged in plumbing and heating contracting until 1927, when he became city engineer and water superintendent at Carroll, Iowa. He formed his present business connection in 1936.

He joined AWWA in 1928 and, in 1948, received the Fuller Award from the Kansas Section, of which he has been secretary-treasurer since 1949. He is also a member of FSIWA.

Missouri—Herbert O. Hartung, superintendent of production, St. Louis County Water Co., St. Louis. Born at Kansas City, Mo., in 1909, he received a degree in civil engineering from the University of Kansas in 1930 and did graduate work at Washington University. After a year as an engineer with Burns and McDonnell, Kansas City, he joined his present firm in 1931. His assignments have included control laboratory operations, pilot plant studies, pumping problems and control, new plant design and construction, and plant management. He is a registered professional engineer.

An AWWA member since 1938, he received the Fuller Award in 1953. The AWWA committees on which he has served include Control of Chlorination, Disinfection of Mains, and Suspended-Solids Contact Basins. He was chairman of the Water Purification Division in 1951. He belongs to APHA and the Missouri Water and Sewerage Conference.

North Carolina—Walter M. Franklin, general superintendent, Water Dept. & Sewage Treatment Works, Charlotte. Born at Altamont, N.C., in 1905, he received a B.S. in civil engineering (1926) and an M.S. in highway and sanitary engineering (1927) from the University of North Carolina. In 1927 he joined the engineering firm of William M. Piatt (now Piatt and Davis) in Durham, N.C., leaving in 1934 to become chief chemist of the water department of that city. He was appointed superintendent of water and sewage treatment plants at Charlotte in 1936 and assumed his present position in 1947, after serving with the US Army Sanitary Corps in 1943–46.

An AWWA member since 1935, he was vice-chairman of the North Carolina Section in 1946–47 and chairman in 1947–48. He received the Fuller Award in 1943 and the Maffitt Member Cup Award in 1939, 1940, and 1941. Other organizations in which he has been active include FSIWA (North Carolina Section past chairman and director), the North Carolina Water Works Operators Assn. (chairman, board of examiners), and the North Carolina Sewage Plant Operators School (member, board of examiners).

Ohio—C. E. Beatty, sales representative, Buffalo Meter Co., Middletown. A life-long resident of Middle-

(Continued on page 42 P&R)

WORTHINGTON-GAMON

WATCH DOG

WATER METERS

The meter used by thousands of municipalities in the U. S.

"Watch Dog" models . . . made in standard capacities from 20 g.p.m. up: frost-proof and split case in household sizes. Disc, turbine, or compound type.

SURE TO MEET YOUR SPECIFICATIONS FOR ACCURACY, LOW MAINTENANCE, LONG LIFE.

Before you invest in water meters, get acquainted with the design and performance advantages which make Worthington-Gamon Watch Dog Water Meters first choice of so many municipalities and private water companies in the United States.

WORTHINGTON-GAMON METER DIVISION

Worthington Corporation

296 SOUTH STREET, NEWARK 5, NEW JERSEY

OFFICES IN ALL PRINCIPAL CITIES

(Continued from page 40 P&R)

Ohio—Beatty

Rocky Mtn.—Turre

town, where he has been prominent in numerous civic activities, he was graduated from Purdue University in 1928 and became district service manager for the Union Gas and Electric Co. From 1932 to 1934 he owned and operated a concrete-block manufacturing concern. He held the posts of city electrician and superintendent of alarms and signals from 1934 to 1941, when he was made assistant water works superintendent. He was named superintendent the following year and was appointed director of utilities when that office was created, in 1945. In 1949 he resigned to take up his present position.

Joining AWWA in 1941, he was elected trustee of the Ohio Section in 1949 and chairman in 1952.

Rocky Mountain—George J. Turre, sanitary engineer, Board of Water Commissioners, Denver. Born at St. Louis in 1894, he attended the Colorado School of Mines in 1917–18. Before becoming assistant chemist for the Denver water board in January 1919, he was a sales engineer for the Denver Fire Clay Co. and a chemist for Root and Simpson, assayers and chemists. During his service with the water board he has held the positions of chemist (1926–35), superintendent of filtration (1935–41), and sanitary engineer (since 1941).

He joined AWWA in 1927, served as chairman of the Rocky Mountain Section in 1943, and was an officer of the Water Purification Division in 1941–47. He represented the section on the National Committee for Utilities Radio in 1948 and was a member of the Goodell Prize Committee in 1953–54. His other organizational affiliations include the American Chemical Society and the Colorado Society of Engineers.

Southeastern—Sherman Russell, superintendent of water purification, Atlanta Water Works. Born in Pickens County, Ga., he saw service with the US Marine Corps in 1917–19. In 1921–25 he attended the Georgia Institute of Technology. Since 1928 he has held various positions with the Atlanta Water Works, and was named to his present post in 1947.

He became an AWWA member in 1943, was chairman of the Southeastern Section in 1950–51, and received the Fuller Award in 1954. Other organizations with which he is affiliated are the Georgia Water and Sewage Operators School (past chairman) and the Georgia Water Use and Conservation Committee.

Virginia—E. C. Meredith, assistant director, Div. of Engineering, Virginia Dept. of Health, Richmond. Born in

S.E.—Russell

Virginia—Meredith

(Continued on page 44 P&R)

For water free of turbidity and color

Check these Advantages of GENERAL CHEMICAL "ALUM"

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- ☐ Gives effective floc formation over wide pH and alkalinity conditions
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Always the same high quality and uniformity—always meeting the most rigid chemical and physical specifications—General Chemical Aluminum Sulfate is preferred by most water men. That's why General Chemical's "Alum" is America's most widely used coagulant for the removal of turbidity, color and other impurities from water.

And, General's nationwide network of producing and shipping points assures you of ready availability. For your needs . . . you can be *sure* when you specify General Chemical "Alum."

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(Continued from page 42 P&R)

W.Va.—Staub

Wis.—Zufelt

Hanover County, Va., in 1907, he holds a B.S. in civil engineering (1930) and an M.S. in sanitary engineering (1932) from Virginia Polytechnic Institute. He has been with the state health department since 1931, except for a year as assistant professor of sanitary engineering at Virginia Polytechnic Institute (1935–36) and 3 years' service with the US Army Corps of Engineers (1943–46). He was appointed to his present position in 1946.

An AWWA member since 1937, he was chairman of the Virginia Section in 1950. He is also associated with FSIWA, the Virginia Industrial Wastes and Sewage Works Assn. (former director), and the Conference of State Sanitary Engineers.

West Virginia—William S. Staub, vice-president and chief engineer, West Virginia Water Service Co., Charleston. Born at Romney, W.Va., in 1910, he was graduated from West Virginia University in 1934, joining his present firm in that year as plant manager. He held the post of engineer from 1935 to 1941 and, after a tour of duty as lieutenant colonel in the US Army Corps of Engineers, became chief engineer of the company in 1946. He was named vice-president in 1948.

He became an AWWA member in 1934 and served as West Virginia Section chairman in 1951. He belongs to the West Virginia Society of Professional Engineers (past president, Charleston chapter) and the West Virginia Sewage and Industrial Wastes Assn.

Wisconsin—Jerome C. Zufelt, superintendent, Sheboygan Water Dept. Born at Sheboygan in 1903, he received a degree in civil engineering from the University of Wisconsin in 1926 and is a registered professional engineer in that state. He joined the consulting firm of Consoer, Older, and Quinlan, Chicago, as an assistant engineer in that year. In 1929 he took a position with the Jerry Donohue Engineering Co., Sheboygan, and in 1930 he was appointed engineer in charge of pumping and filtration in the water department of that city. He has been superintendent since 1939.

He became an AWWA member in 1931 and served as trustee (1940), vice-chairman (1941), and chairman (1942) of the Wisconsin Section, receiving the Fuller Award in 1948. His AWWA committee memberships include the Task Group on Fluoridation Materials and Methods (chairman, 1950 and 1954) and the Safety Manual Committee.

Mfr.—Orth

Manufacturer—Robert F. Orth, manager, Transite Pipe Dept., Johns-

(Continued on page 46 P&R)

don't stick your neck out!
When you know cast iron pipe lasts over 100 years—
When you know cast iron pipe is inherently stronger, tougher, uniform in quality—
When you know cast iron pipe has a matchless history of low installation and low maintenance costs—
When you know cast iron pipe's long, trouble-free life saves millions of tax dollars—
When you know cast iron pipe is the standard conduit for carrying water underground—
When you know all these facts, then you know, too, it's unwise to experiment with substitutes. Experienced city officials look to cast iron pipe's proven record as their guarantee of a sound, long-term investment. And, for over three-quarters of a century, they've learned to look to Clow for the finest cast iron pipe made. They have found consistent high quality coupled with excellent service and fast delivery. They've found centrifugally cast Clow Cast Iron Pipe offers the greatest economy, and it meets all currently approved specifications now in existence. Write today for complete information.
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Iowa Valve Co., Oskaloosa, Iowa
CAST IRON

(Continued from page 44 P&R)

Manville Sales Corp., New York. Born at San Antonio, Tex., in 1901, he was graduated from Texas A. & M. in 1923 with a degree in mechanical engineering. He joined Johns-Manville as an acoustical engineer in 1924 and has subsequently held the positions of salesman, architectural representative, staff manager of the Transite Pipe Dept. for the St. Louis District, and manager of the company's public utility and electrical departments. He is a past president (1951) of the Water and Sewage Works Manufacturers Assn.

Not just the humidity, but the heat as well is now to be measured water-wise. That was the decision of the Tenth General Conference on Weights and Measures at Sevres, France, last fall, when standards experts from 35 different countries agreed to redefine the absolute temperature scale in terms of the triple point of water—the point at which ice, water, and water vapor can exist in equilibrium. And if the establishment of this single fixed reference point, by definition, at 273.16 degrees Kelvin doesn't immediately make you feel a little cooler, just relax and enjoy the augmented indispensability of your product.

Harry C. Ohge of Indianapolis has been appointed Hungerford & Terry representative in Indiana. Mr. Ohge has had 28 years' experience in the water treatment business and has been located in Indiana for the past 25 years.

(Continued on page 48 P&R)

AMONG WATER WORKS MEN

THE HEAVY-DUTY
ELLIS
PIPE CUTTER
IS *BEST*

FOR CUTTING LARGE SIZES OF PIPE

No. 01 Cuts Pipe 4" to 8"
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Write for circular and price list No. 35J, on our complete line of pipe cutting tools.

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Anthrafilt Offers Many Advantages Over Sand and Quartz

- DOUBLES length of filter runs.
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- KEEPS Filters in service over longer periods.
- INCREASES Filter output with better quality effluent.
- GIVES better support to synthetic resins.
- PROVIDES better removal of fibrous materials, bacteria, micro-organic matter, taste, odor, etc.
- IDEAL for industrial acid and alkaline solutions.
- EFFECTIVE filtration from entire bed.
- LESS coating, caking or balling with mud, lime, iron, or manganese.

Write for further information, test samples and quotations to:

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He knows how to take the Debbil out of the Sun!

Old Sol . . . Devil or Saint can be . . . but to the municipality with a properly engineered water supply system, the Sun can be all Saint.

Prolonged droughts that result in water scarcity can be disastrous to a city's growth to say nothing of its citizens' health and welfare.

Layne—the world's largest developer of ground water resources—has accumulated through nearly three quarters of a century knowledge of inestimable value to municipalities large and small. It is free for the asking.

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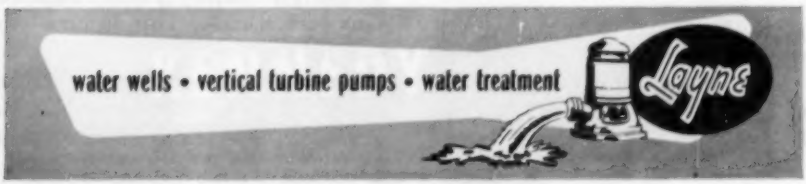

(*Continued from page 46 P&R*)

It's Sudsy Williams now, Michigan's governor—the erstwhile Soapy—having gotten himself into deep water in the winning of the 1955 Layne Award. Shown below accepting the award from President Edward J. Rogers of Layne-Northwest Co. (left) and President and Chairman of the Board J. I. Seay of Layne & Bowler, Inc. (right), Governor Williams hardly seems in a lather. But then, ground water—which is Layne's business—is often rather hard. The fact that the

water which makes Michigan a "Water Wonderland," which the governor won his award for his outstanding work in promoting, isn't ground water and that ground water could hardly be used to make the kind of a "Water Wonderland" that Michigan is, is rather confusing. As long as it's water, though —and the "world's greatest fresh water supply" at that—that Soapy is promoting, we're all for him—and all for Layne, too, for making him Sudsy.

J. D. Crell has been named sales engineer for the New York City Sales Dist. of Graver Water Conditioning Co., New York. Mr. Crell has been with the company for several years.

Another kick coming has postman Robert W. Milligan of Bartlesville, Okla., who put his toe to a troublesome dog on his route and ended up unemployed. It took the Civil Service Commission Board of Appeals in Washington to restore him to his job, their basis being that it was permissible for a postman to kick a vicious dog in self-defense. Just how Milligan proved the dog was vicious and that he kicked in self-defense might make an interesting story for water works metermen—not recommended reading though, for more than a little ill will could be expected from the dog lovers among our customers if the practice were adopted. And with more than 50 per cent of our customers in that category, the kickback could be rather disastrous.

On other grounds, too, the quick kick routine seems impractical. In the first place it wouldn't be long before the fast-footed meterman found every dog vicious every time, for there's nothing like an aggressive defense to incite a defensive attack. Estimates of canine population put at 22,000,000 the number of dog pets now kept in the US. That would mean at least 15,000,000 kept by water utility customers and, very conservatively, 10,000,000, by metered customers. Assuming an average of 4½ meter readings per year and a total of 15,000 meter readers, we find that every reader would be making an average of 3,000 kicks per year. As that would give him very little time for reading, it would undoubtedly become necessary to employ a kicker to accompany each reader. This, in turn—well —seems impractical.

Not quite on the subject, Vic Weir, our caninic commentator from St. Louis County, has apparently been frightened

(*Continued on page 50 P&R*)

KENNEDY VALVE gives you the *complete* story on A.W.W.A. Valves . .

Finger-tip facts to help you specify A.W.W.A. Valves . .

KENNEDY has available for you now, a bulletin with complete information on the Kennedy line of A. W. W. A. Gate Valves. You'll find detailed specifications including lists of the wide range of types, sizes, controls, accessories plus connections and gearing arrangements available. Here's excellent reference material combined with important facts you need when ordering or specifying A. W. W. A. Valves. Get your free copy now!

• *Write for Bulletin 106*

THE KENNEDY VALVE MFG. CO. ELMIRA, NEW YORK Est. 1877

VALVES • PIPE FITTINGS • FIRE HYDRANTS

OFFICE-WAREHOUSES IN NEW YORK, CHICAGO, SAN FRANCISCO • SALES REPRESENTATIVES IN PRINCIPAL CITIES

(Continued from page 48 P&R)

by a TV commercial into concocting this verse:

> There's a new food
> That makes dogs beg;
> It has a flavor like
> A meter reader's leg.

At that, some kick is really coming!

Adequate water supply for tomorrow was the theme of the water supply luncheon held May 3 at the Mayflower Hotel, Washington, D.C., as part of the forty-third annual meeting of the US Chamber of Commerce. Ervin L. Peterson, assistant secretary of the US Dept. of Agriculture, spoke on "Water for Our Farms." Kenneth M. Lloyd, secretary of the Mahoning Valley Industrial Council, Youngstown, Ohio, gave a talk on "Water for Expanding Industrial Use." "Water for a Growing Population" was discussed by AWWA Secretary Harry E. Jordan. It is planned to publish these papers in a forthcoming issue of *Willing Water*.

The importance of water these days goes without saying—which is to say that everyone, at last, is saying something about it. It hardly seems possible that less than ten years ago we had to scrape around in every corner of our newspaper to find mere mention of water supply upon which to base our extravagances, when now we haven't time even to read all the lengthy, weighty articles that rate major headlines. Where ten years ago we would have made much of Councilman J.

(Continued on page 52 P&R)

BUILDERS PROPELOFLO is an inexpensive main line meter for totalizing water consumption. Gives dependable, trouble-free service on the job. Meters accurately over wide range — six-digit totalizer shows water use directly in gallons, cubic feet, etc. For complete information on this easy-to-install meter, write to Builders-Providence, Inc., 365 Harris Ave., Providence 1, Rhode Island.

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FOR TOP QUALITY PIPE LINE EQUIPMENT

CHECK POLLARD

For complete, modern pipe line equipment to meet virtually all your needs, be sure to call on Pollard! *More than a century of service and experience are behind each of the one-hundred-twenty-six equipment categories listed in the new Pollard Catalog.*

Hundreds of Pollard customers depend on Pollard for prompt delivery of highest quality pipeline equipment, backed by the most cooperative service in the industry.

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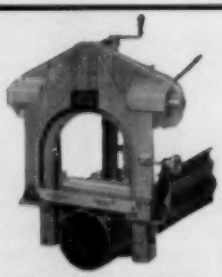
(Continued from page 50 P&R)

Smith's statement in support of a second well for the town of West Lynne, these days we leave all but unreported the unsolicited testimonials of such celebrities as David Lilienthal, who not long ago pronounced water supply the nation's most important problem; we hardly mention the struggles of the Western States over the Upper Colorado River storage project which Senator Clinton Anderson termed "probably the most important single piece of legislation in the history of federal reclamation"; we shunt off to the more dignified pages of the JOURNAL the story of the East's awakening to the problem of water rights. As a matter of fact, we're so overwhelmed with evidence of water's importance that we find arguments unnecessary and extravagance impossible.

The secret of this quick change, of course, is no more mysterious than the sudden realization, inspired by a few particularly dramatic shortages, that all the water that everybody wants isn't always available everywhere. And nothing, it seems, is cheaper than dearth as a means of convincing the public of value. Fed by the growing problem of summer shortages, this new public concern over water supply is a major asset on which water utilities can capitalize, not only in bringing their rates up more nearly to recognized value but in bringing their supplies up more nearly to recognized requirement. To P&R, however, which specializes in obscurities, all this public attention is not asset, but anathema. The importance of water, in other words, henceforth goes without our saying.

(Continued on page 86 P&R)



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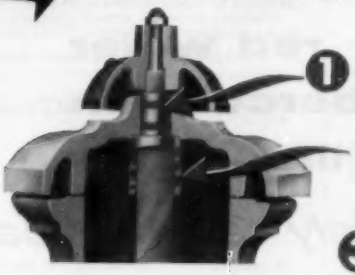

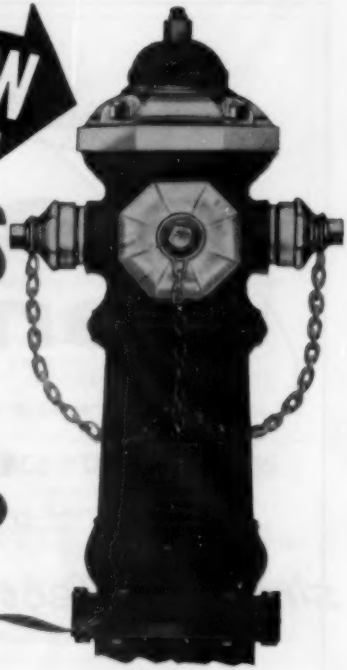

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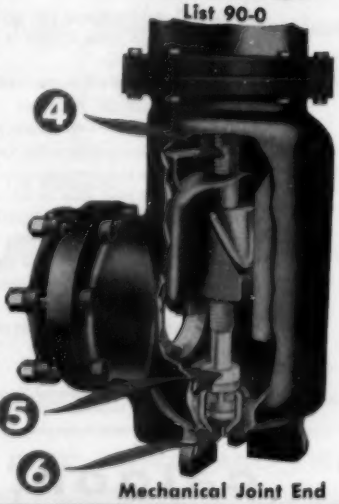

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

CHEMICAL ANALYSIS

Some Notes on the Method of Alkalinity Determination. Y. MACHIDA. Bul. Japan. Soc. Sci. Fisheries (in English), 19:703 ('53). For field work in detn. of alky. of inland and sea water, various modifications were proposed. Simple semi-microburet was described. Mixed indicator prepd. by adding 1 part of 0.1% aq. soln. of methyl orange to 1 part of 0.1% aq. soln. of aniline blue was used; it changed color at pH 4.3 from greenish brown (alk. color) to reddish violet (acid color). Employment of both buret and indicator gave agreeing results. Relation between alky. and CO_2 components in natural waters discussed.—*CA*

Detection of Barium and Strontium in Water Analysis. K. E. QUENTIN. Z. Lebensm.-Unteraasch. u. -Forsch (Ger.), 99:85 ('54). Method for small amts. Ba and Sr in natural waters entails sepn. from troublesome materials and concn. of alkali earth ions. Tippable column app. of Wickbold with synthetic ion-exchange resin adsorbent is used. Elution is with 10% HCl soln.; eluate is evapd. and residue can be paper-chromatographed. With heavily mineralized water, eluate can be treated with NH_4OH and $(NH_4)_2CO_3$ to ppt. alkali earths, which are filtered out, dissolved in HCl, and paper-chromatographed. Developing liquid is soln. of 500 mg NH_4CNS and 5 ml concd. HCl in 95 ml MeOH. Sr gives bright-red spot or streak. In presence of Ba, reddish violet spot occurs between color due to Sr and start. Quant. data obtainable by comparisons with Ba and Sr standards developed similarly.—*CA*

The Importance of the Presence of Alkali Bicarbonates, Nitrates, and Phosphates, Commonly Disregarded as Ground Water Constituents. A. SPLITTGERBER. Wasserwirtschaft. Tech. (Ger.), 4:162 (54). It is proposed that 3 named constituents be included in customary water anals. as aid in water evaluation. Bicarbonate content of waters from 4 main geologic regions of Germany given, and correlations drawn between geology and bicarbonate content. Of 1,844 samples tested, 902 contained more than 0.5 mg/l nitrate. Nitrates, though not ordinarily harmful ingredients, are indicative of previous poln. by org. matter. Moreover, phosphates indicate possibility of ground water leaching through submerged strata of slowly decompg. protein.—*CA*

(*Continued on page 64*)

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54				192	190	580	230	225	695	267	260	810	303	295	925
60				214	170	525	255	200	625	296	235	730	336	265	835
72							309	170	520	359	195	605	408	220	695
84							357	145	450	412	170	520	470	190	595
96	The recommended minimum wall thickness is approximately the pipe diameter divided by 165. Pipe having this diameter-thickness ratio, when backfilled and properly tamped, will withstand any depth of cover. For buried pipe, the recommended minimum thicknesses are shown immediately above heavy black line.									470	145	455	539	165	520
108													*611	150	465
120													*669	135	415

Based on use of ASTM A-283, Grade B Steel, 50,000 psi min. ultimate tensile strength.
*Under certain conditions stiffeners may be required to reduce deflections.

$P = \frac{t \times T_s}{r}$

P = internal pressure, psi
t = thickness, in.
T_s = allowable unit stress = 60% x 27,000 (yield point) = 16,000 psi
r = radius of pipe, in.

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Spinks

(*Continued from page 62*)

Determination of Calcium and Magnesium in Mineral Waters. M. KOBROVA. Chem. Listy. (Czech.), **48**:1255 ('54). To det. Ca treat 50–100 g mineral water with 1–2 g Na tartrate and known quant. of complexon III soln., expel CO_2 by boiling, dil. soln. with 250–300 ml H_2O, add murexide and 2 g NaOH, and retitrate excess complexon with Ca soln. (color change from violet to red). Det. Mg content from difference of consumption of complexon soln. in titration to murexide and to Eriochrome black T. Dil. 50–100 g mineral H_2O with 300 ml distd. H_2O and, after heating at 60°, add 5 ml of buffer soln. (prepd. from 54 g NH_4Cl and 250 ml 35% NH_3 dild. to 1,000 ml with H_2O) for each 100 ml soln., Eriochrome black T, and known excess of complexon. Excess complexon retitrated with Ca after short heating to 90°–100°.—*CA*

Determination of Free Chlorine and Chloramines in Water. W. SZYPOWSKI. Przemysl Chem. (Poland), **9**:122 ('53). Amt. of free chlorine and chloramines is detd. with KBr and dild. soln. of methyl orange (460 mg/l) or methyl red (254 mg/l); 1 ml of this soln. equals 1 mg Cl. 100 ml H_2O contg. 0.5 ml H_2SO_4 is titrated with pigment soln. until color is obtained; amt. of Cl is calcd. Then 0.5 ml KBr added and, if discoloration occurred (presence of chloramines), soln. is further titrated with pigment soln. until pink color is obtained. Amt. of Cl in chloramine is found by multiplying ml of pigment soln. by 0.89 for methyl orange or 0.75 for methyl red. This titration gives results 25–33% lower, owing to escape of Cl. In second exact titration, 25% more pigment soln. is added than in preliminary one, and acidified with H_2SO_4; soln. is then titrated or back-titrated to obtain amt. of Cl. Subsequently, KBr added and titrated to obtain amt. of Cl in chloramine. —*CA*

Determination of Chlorine in Water. Suggested Use of 3,3′-dimethylnaphthidine. R. BELCHER, A. J. NUTTEN & W. I. STEPHEN. Anal. Chem., **26**:722 ('54). 3,3′-dimethylnaphthidine was found superior to benzidine and *o*-tolidine as reagent for detn. of Cl in water because of its higher sensitivity, stability of color, and ease of color measurement. On simple, single-cell, direct-reading colorimeter, accurate results were obtained with as little as 0.05 ppm Cl. To

(*Continued on page 66*)

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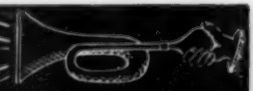

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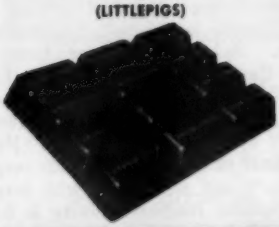

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(*Continued from page 64*)

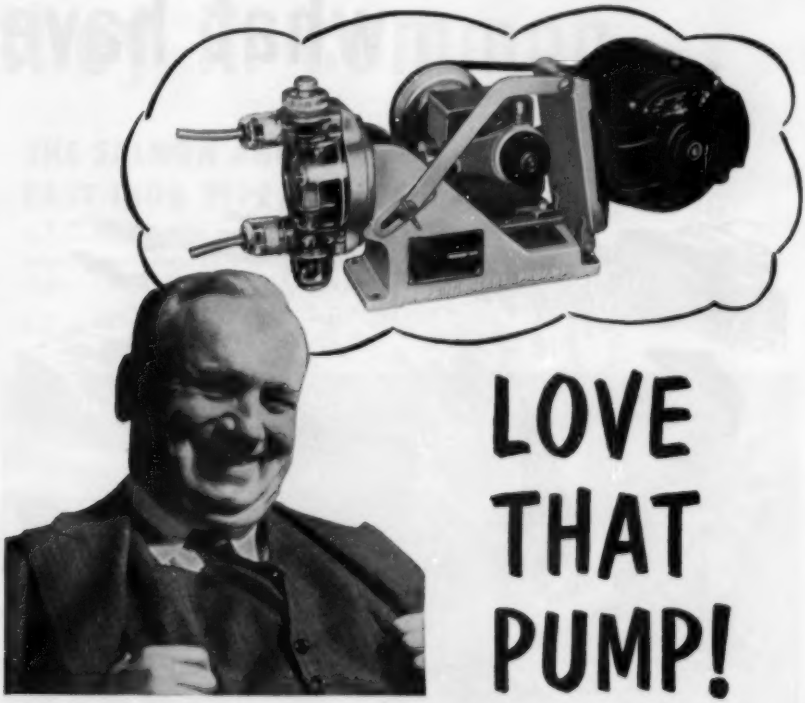
100 ml of sample, add 1 ml 0.1% soln. 3,3'-dimethylnaphthidine in glacial AcOH and mix. Purple-red meriquinonoid product is formed. Allow to stand 5 min and then read color intensity photoelectrically with green filter. Compare with values obtained from standard calibration curves.—*CA*

The Action of the Fluoride Ion on Colored Inorganic Salts. M. E. Maraboli. Tesis Quim., Univ. Chile, 3:147 ('51). Colorimetric procedures were investigated for detn. of F^- in drinking water. These are based on discoloration of standard solns. by F^-. Zr-alizarin method is superior to others because of bright color change. Other useful procedures are Mo thiocyanate, $Fe(SCN)_3$, and vanadate-H_2O_2 methods. Sensibilities are 0.005–0.01 mg of F. Silico- and phosphomolybdate methods are less sensitive. Discoloration of $MnPO_4$, "triferric dihydroxy hexacetate," and $Cu_2Fe(CN)_6NH_3$ was studied without yielding practical methods. Better values are obtained if distn. to sep. F^- from interfering ions is carried out with NaClO (Sanchis' method) rather than with $HClO_4$.—*CA*

Spectrophotometric Determination of Traces of Iodine in Water. J. Pirotte & A. Berger. Bul. Centre Belge Étude et Document. Eaux (Belg.), No. 13 (Pt. III), p. 177 ('51). 1–50γ of I is detd. with error of 10% by detn. of rate of reduction of Ce^{4+} by As^{3+} catalyzed by I. Reaction was detd. to be 1st order with respect to I and Ce^{4+}, and zero order with respect to As^{3+}. Specific reaction velocity is 0.023 ± 0.003 at 25° (pH = 0.8). In detn., 25 ml of $2 \times 10^{-5}M$ ceric sulfate in $1N$ H_2SO_4, 25 ml of neutral 10^{-3} sodium arsenite, and 50 ml of sample are mixed and placed in thermostat at 25° ± 1°. After 5 and 15 min absorption of Ce^{4+} is detd. at 375 mμ. Ag, Hg, and Os interfere, while F and Br (up to 0.5 mg/l) and Cl (up to 1 g/l) do not interfere.—*CA*

Experimental Part of the Study of the Determination of Lead. V. Royer. Bul. Centre Belge Étude et Document. Eaux (Belg.), No. 13 (Pt. III), p. 170 ('51). Percentage error in detn. of Pb was 0.5–5.8 for amts. between 10 and 35 γ/l. Water from various sources in province of Liege showed 60–500 γ/l Pb after standing

(*Continued on page 70*)



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pipe FOR MODERN WATER WORKS OPERATION

(Continued from page 66)

for 8 hr in Pb pipes. After purging, Pb content fell to 14–20 γ/l.—*CA*

Spectrophotometric Determination of Lead in Water. A. Berger & J. Pirotte. Bul. Centre Belge Étude et Document. Eaux (Belg.), No. 13 (Pt. III), p. 160 ('51). Dithizone method used. Methods of calcn. given when absorption measurements are made in $CHCl_3$ at 520 mμ (max. absorption of Pb dithizonate) and 610 mμ (max. absorption of dithizone) or at 520 mμ and at 553 mμ, where absorption is independent of relative concns. of Pb dithizonate and dithizone.—*CA*

Influence of Minium-Painted Tanks on the Lead Content of Water. J. Just & B. Koziorowski. Gaz, Woda i Tech. Sanit. (Poland), **26**:315 ('53). Purpose of work to establish whether water kept in minium-painted tank takes up Pb from coating, in what degree, and for how long after painting. Expts. were conducted in Fe containers of 2–6.5-liter capacity. They were painted with 2 layers of minium paint prepd. by mixing 20 parts by wt flax oil with 80 parts of minium (96.95% PbO). Two different kinds of oil used: coml. one and one prepd. in lab. by addn. 0.75% MnO_2 and 7 hr mixing at 250°. Expts. done with 2 different waters: city water and city water to which chlorides and nitrides added in proportions of 250–5 mg/l Cl and 0.3–25.0 N. Every container, after rinsing, filled with water and left for 72 hr at room temp. (16–25°). Every 24 hr Pb, O consumed, and smell detd. on sample. After 72 hr container emptied, rinsed, and filled up again for another 72-hr cycle. Procedure repeated 23 times for each tank. Results indicated that amt. of Pb contamn. depended on qual. of oil used in minium paint. Pb content in city water fell down to required std. (< 0.05 mg/l) after very long time: 22 cycles for containers painted with coat prepd. in lab., and more than 23 cycles for those painted

(Continued on page 72)

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(*Continued from page 70*)

with coml. coat. O consumed increased (up to 2 and more mg/l of O) up to 16–17 cycles and then fell down; paint smell disappeared after 8 cycles.—*CA*

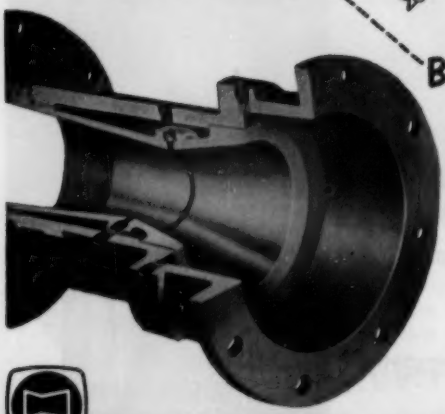
Continuous pH Recorder for Water Analysis. H. HIENTZSCH. Wasserwirtschaft Tech. (Ger.), **4**:332 ('54). Continuous pH-recording instrument in 3 parts is described, comprising electrode assembly and housing, current amplifier, and ink recording app. Measuring cell is glass electrode filled with standard CH_3COONa soln. and Ag-AgCl reference electrode. H_2O continually passes through electrode vessel. Recording app. has scale calibrated from 0 to 300 mv and pH 3.4 to 8.8, throughout which range, pH-mv relation is linear.—*CA*

Determination of Sulfate in Natural Waters by the Ion Exchanger and Ethylenediaminetetraacetic Acid. K. ISAGAI. J. Chem. Soc. Japan, Pure Chem. Sec., **75**:613 ('54). Natural water contg. sulfate ion is passed through cation exchanger column, and $BaCl_2$ is added to filtrate. Ppt. filtered and excess Ba ion detd. by titrating with ethylenediaminetetraacetic acid in presence of Mg. From 10 to 2,560 mg/l sulfate is detd.—*CA*

Problem of Significance and Determination of Organic Matter in Water. H. IVEKOVIC & A. GERTNER. Acta Pharm. Jugoslav., **3**:46 ('53). Survey of signif. and detn. of total org. matter in water is given. In detn. of total org. matter, it is necessary to prove presence of material which indicates fresh contamn. of water as well as to det. percentage of N. Instead of proving contamn. of fecal origin proof of presence of easily oxidizable matter is sufficient. It is also shown that water contg. org. matter fluoresces in ultraviolet light. Matter which is sparingly oxidizable indicates contamn. of older date whereas matter which is easily oxidizable shows fresh contamn.—*CA*

(*Continued on page 74*)

When To Use



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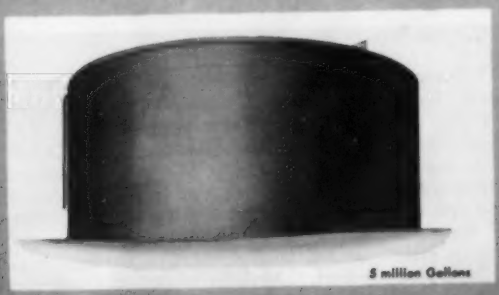

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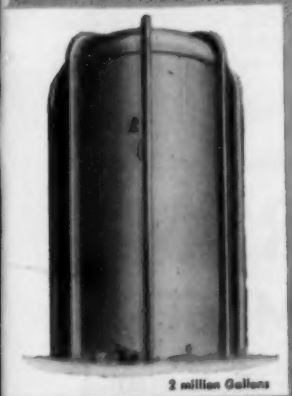

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(Continued from page 72)

The Relative Difference in H_2O^{18} Content of Natural Water. P. BAERTSCHI. Helv. Chim. Acta (Swiss), **36**:1352 ('53). 91 samples examd. included rain, surface water, subterranean water, and water obtained by dehydrating minerals. Relative difference in ratio of O^{18}/O^{16} was detd. by mass spectrograph with Basel tap water as standard. Variation, expressed in percentage of ratio in standard, ranges from −7.1 to +18.0. This variation is discussed in terms of source. —*CA*

SOURCE OF SUPPLY—QUALITY

Water Resources of the Pittsburgh Area, Pennsylvania. M. NOECKER, D. W. GREENMAN & N. H. BEAMER. US Geol. Survey Circ., No. 315, p. 1 ('54). Many chem. analyses of ground waters, river waters, and water supplies of various cities are given. Use in 1951 was over 3,000 mgd, more than 90% for industrial purposes. Acid mine drainage is serious pollution problem.—*CA*

Report on Investigations Concerning the Influence of Vegetation on the Quality of Drinking Water From Dunes. R. WIND. Mededeel. Inst. Toegepast Biol. Onderz. Nat. (Neth.), **12**:1 ('52). Investigations were made on influence of vegetation on quality of drinking water from dunes in Netherlands, particularly with view of ascertaining whether afforestation would have beneficial or harmful effect. Analysis of water retained in soil above capillary zone just below zone in which most of roots occurred gave reliable estimate of influence of vegetation on composition of underground water. Method by which sufficient quantities of retained water could be obtained was devised. Water from soil of pine and oak forests showed higher values of color, $KMnO_4$ absorption, HCO_3 ion, hardness, and chloride content than water from soil with natural, low vegetation. Influence of pine and oak forests on quality of water must, therefore, be considered unfavorable. Pine forest, however, on dunes rich in lime is not always less favorable than forest consisting of oak and birch. First

(Continued on page 76)

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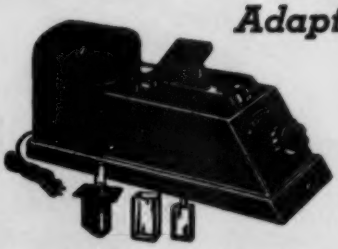

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(*Continued from page 74*)

type of forest gave lower figures for color and $KMnO_4$ absorption but higher figures for chloride content and hardness. Moisture content is strongly influenced by vegetation. In winter moisture content of soil under forests was higher than under natural dune vegetation. There was difference between seasonal fluctuations of moisture content of soil under pine forest and under oak forest. Investigations produced some data which cannot yet be interpreted. It was clear, however, that pine forest retains salt from wind passing through it and transmits this to ground water, with result that ground water under forest contained more chloride than that under dunes with natural, low vegetation to seaward side of forest. Oak forest with undergrowth of elder showed very high nitrate figures for retained water. Only few types of vegetation were included in investigations, and these were all on dunes poor in lime.—*PHEA*

Water Supply From Mine Shafts. C. V. Brown. J. Inst. Water Engrs. (Br.), **8**:119. Ordinarily water from collieries of North Staffordshire coal field unsuitable for supply purposes. It was possible, however, to develop satisfactory supplies from two sites where mine shafts, dug for producing coal, had been abandoned. Water from local colliery shafts had 1,930–4,555 ppm residue. Water from abandoned shafts had 270, 260, and 235 ppm residue. Where shafts had been abandoned, bacterial condition was good and water was satisfactory source for public water supply.—*CA*

Geochemical Aspects of Problems Concerning Spring and Thermal (Mineral) Waters. R. Pieruccini. Ber. Intern. Balneol. Kongr. ISMH. Deut. (Ger.), p. 130 ('52). Importance of principal and of accessory constituents of spring and thermal waters is discussed in relation to interactions between these waters and country rock material in which they circulate. Reaction equil. are detd. by detg. concns. of natural solns. Pelitic sedimentation in sea water, of abyssal and littoral character, is detd. by reaction

(*Continued on page 78*)

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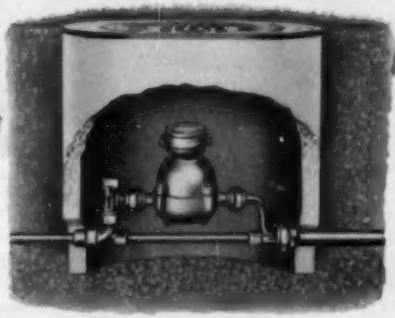
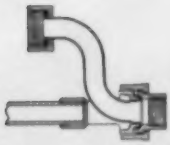


ON WATER METER INSTALLATIONS WITH THE

FORD LINESETTER



Section view of tube end assembly showing double-purpose connection for iron or copper.

Type VEA Linesetter 45° angle inlet for copper. Double Purpose Coupling Outlet.

The Ford Linesetter is as easy to install as a new pipe union in all types of shallow meter settings, with all kinds of meter boxes.

- Service line connections and settings are compact, and the valve is easy to reach.
- Costs for labor and maintenance go down.
- Ford set meters are easier to read, easier to change, stay cleaner, thus receiving better care and earning more revenue over a longer lifetime.

Send for free Ford catalog No. 56

Manufactured by

THE FORD METER BOX COMPANY, INC.

Wabash, Indiana

FOR BETTER WATER SERVICES FORD

(*Continued from page 76*)

equil. complicated by trace elements, which can be used in study as "guide elements." Geol. factors and knowledge of lithological type of country rocks permeated by waters are important. Spring waters are in state of chem. equil. when coming to surface, but contact with organisms immediately changes these states of equil. This is basis for scientific investigation of therapeutic actions of mineral waters. Trace elements, Co, Mn, Sr, Zn, Pb, Ag, V, and Cr, exert stimulating or toxic effects on organisms. Distr. and migration of trace elements can be studied by spectrographic methods, often combined with preliminary enrichment of guide elements in question, e.g., by copptn. Accuracy of detns. may easily be ±0.5 γ/l, or even better. Also, inorg. deposits, calcareous, siliceous, and ferrugineous pptns. from waters, must be studied. Systematic investigations of this kind are important for geochemistry and biogeochemistry, combining special crystallochem. effects of "camouflage" with those of "ubiquity."—*CA*

Desalting of Bore Water by Solar Distillation. B. W. Wilson. Divisional Rpt. No. 71, Council Sci. Ind. Research Organization (Australia). Solar distn. method of Telkes studied under Australian conditions, with 3% NaCl in still of 5-sq ft tray area. Batch operation more reliable than continuous. On clear days, water yields were 1.29–1.46 lb/sq ft/day for Dec.–Jan. (insolation approx. 30,000 BTU/sq ft). Hourly efficiency at noon (detd. graphically) was 62–69% and overall daily efficiency, 50%. Estd. avg daily still yields (lb/sq ft/day) for other localities are Adelaide, 0.79; Alice Springs, 0.93; Darwin, 0.84.—*CA*

Studies of Waste Water Reclamation and Utilization. A. F. Bush & S. F. Mulford. Pub. No. 9, State Water Poln. Control Board, Sacramento, Calif. Report covering detn. of: [1] relationship of underground water poln. to methods and rates of spreading and percolation of reclaimed waste wa-

(*Continued on page 80*)

ELEVATED TANKS

For almost a century Cole elevated tanks have been helping provide uniform water pressure, fire protection and adequate water reserve in scores of American cities.

Capacities 5,000 to 2,000,000 gallons—with hemispherical, ellipsoidal or conical bottoms. Also flat-bottom tanks for standpipe storage. Correctly built in accordance with AWWA specifications.

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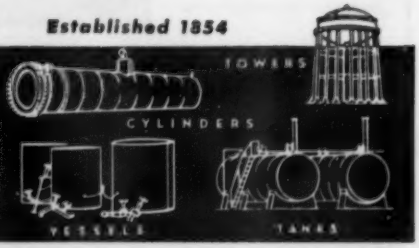

AMERICAN FERROFILTER

Installation of "American" Ferrofilters and "American" Double Suction Split Case Pumps at Columbus, Indiana Water Plant • Consulting Engineers, Henry B. Steeg & Associates, Inc., Indianapolis, Indiana.

Independent tests and observations of "American" Ferrofilter operation at the Columbus, Indiana water plant show reduction to .15 ppm on 1.8 MGD loading/filter. (Units are rated 1.5 MGD.) Even on excess loading of 2.5 MGD/filter with all six wells on, iron reduction was still getting down to .2 ppm.

The "American" Ferrofilter is a contact filter that offers many advantages as a complete process or as a preliminary process—for the removal of iron, manganese, carbon dioxide, and other dissolved gases and odors, in one simple operation. It combines fundamental processes and design in an effective and economical treatment plant. Utilizes fine media in open aeration. Can be backwashed to remove any excess accumulation, leaving the filter in condition for another cycle. In most cases, it eliminates the need for a sedimentation tank or sand filter.

Write for Bulletin No. 252B or for specific information relating to your application.

Utilize our experience in engineering design and manufacture of field-proved equipment and pumps for water and waste treatment.

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Pumping, Sewage Treatment, and Water Purification Equipment
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(Continued from page 78)

ters; [2] effects on underground water poln. of increasing rates of percolation of reclaimed waste waters by cropping and vegetation, additives to top layers of soil, forced irrigation, or other means; [3] either degree of contamn. or poln. (or both) of variety of truck crops on maturity where reclaimed waste waters from various sources, including sewage and industrial wastes, have been applied to crops and spreading areas by flooding, spraying, or other means; [4] extent of odor or other nuisances which may result from such procedure. Report is replete with charts and tables, not only for California but for other states. Authors conclude that sewage is resource in many localities and is used in some parts of United States for recharge of aquifers. Distance contamn. and poln. will travel through soils appears to be function of percolation rate. Initial rates of percolation appear to range from 0.5 ft to 200 ft/day for spreading operations and depend on various factors. Effluents from sewage plants providing only primary treatment, when spread on crops of land, appear to produce high initial coliform bacteria counts in surface soil and on foliage wet by effluent. Authors recommend waste water should be considered water resource and further studies be made of rates of percolation and degree of treatment required to handle poln. load, as well as to remove salts (particularly sodium and boron) from sewage and industrial wastes. Useful references and bibliography are furnished.—*PHEA*

For Organic Iron and Color Removal—Algae Control—Regeneration of Iron Removal Filters—

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Supplied to industry and municipalities in various grades and meshes to fit the particular operation efficiently and economically in combating many water problems.

WRITE FOR QUOTATIONS

CARUS CHEMICAL COMPANY

Box JAW LaSalle, Illinois

Reclamation of Saline Waters by Electrodialysis Shows Promise. R. Eliassen. Civ. Eng., **44**:366 ('54). For centuries men have been dreaming of extracting plentiful supplies of fresh water from ocean and other brackish surface and underground sources. Salt extraction methods are known but costs have prevented construction of extensive reclamation works. Construction costs have been estimated to be approx. $7,000,000/mgd capacity and operation costs from $0.50 to $1.00/1,000 gal of fresh water produced. New process known as electrodialysis has been developed which may provide more economical method of producing fresh water from saline. Method consists of using electrical forces to pull cations and anions through semipermeable ion-selective membranes leaving water behind. Process utilizes semipermeable properties of certain types of plastic membranes which permit only passage of positive ions in parallel with other types of plastic membranes which will only pass negative ions. Estimate has been made of cost of process for converting sea water into irrigation water. For plant having daily capacity of 3,000 acre-ft, construction cost would be $280,000/mgd capacity, and operating cost $0.32/1,000 gal.—*PHEA*

Water Demineralization by Electrodialysis. E. Wegelin. Bul. Centre Belge Étude et Document. Eaux (Belg.), No. 21, p. 182 ('53). Review of principles. Particular attention paid to comparative costs of distn., ion exchange, and electrodialysis. With crude water contg. electrolytes equiv. to 1,000–5,000 ppm Cl^- to be reduced to 300 ppm or less, electrodialysis is preferred. With sea water in range of 20,000 ppm Cl^-, costs of electrodialysis and distn. are about same with appreciably lower initial investment for electrodialysis plant.—*CA*

(Continued on page 82)

NEW! a flow meter with no flow restrictions!

FOXBORO
MAGNETIC
FLOW METER

This premium-performance flow meter *magnetically* measures the flow rate of virtually any liquid except hydrocarbons—without adding any pressure drop to the line! It connects into the line like an equal length of pipe... has no pressure taps or moving parts... needs no maintenance. Overall accuracy is better than 1% of range *over entire scale* . . . even on rock-and-acid or sand-and-water slurries!

The Foxboro Magnetic Flow Meter operates on the same principle as a power generator. Liquid passing through its magnetic field generates voltage proportional to average velocity. This voltage is recorded directly in flow units by a Dynalog Recorder or Controller.

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FOXBORO Reg. U. S. Pat. Off. Foremost in FLOW METERING

FACTORIES IN THE UNITED STATES, CANADA, AND ENGLAND

(Continued from page 80)

FOREIGN WATER SUPPLIES —GENERAL

Water Supply and Sewage Disposal in the Industrial Ruhr District [Ger.]. H. ROHDE. Planographer Pamphlet (Ger.). Ruhrverband, Essen, 12 pp. Ruhr R. supplies water to large industrial area for industry and drinking. In '52 water taken from Ruhr was 1.023×10^9 cu m (270,277 mil gal). Of this, 20% was used for approx. 4,500,000 population, and 80% for industry. About half of water from Ruhr watershed is used for Ruhr population and industry. Other half is pumped to different areas, including Emscher. Owing to increasing demand (from 450 million cu m/yr in 1922 to 10^9 cu m/yr in '52), strict regulations are enforced relating to reservoir construction, both presently and in future. Program of treatment of sewage and industrial wastes, rigidly enforced, includes diversion of sewage and effluents from Ruhr to Rhine R. Because of increasing poln. load on Rhine, treatment works are now planned and under constr. For treatment in Ruhr area, modern high-rate trickling filters are planned, in accordance with German practice. In purif. of Ruhr itself, treatment of industrial wastes is essential, either by Ruhr Assn. or industry, particularly to reduce amt. of phenol produced and remove acid pickling waste liquor. Over years, Ruhr Assn. has built, and is now operating: 80 biol. and mechanical treatment plants for sewage of towns; 25 sewage-pumping stations and high-water pumping stations; 4 reservoirs in course of river to intensify biol. self-purif. of river; 48 treatment plants for industrial wastes. Total length of collections for sewage and industrial-waste effluents built by Ruhr Assn. is, roughly, 250 km (155 mi). Reservoirs are available for recreation. Locks are provided to pass 300-ton barges from Rhine, with proposed lockage for 12,000-ton barges. Similar projects have been developed on Emscher R. Paper is accompanied by diagram and map.—*PHEA*

The Condition of the Ground Water Around Cologne, Germany. E. HOLLER. Gas- u. Wasserfach (Ger.), **95**:16:497 ('54). Cologne lies in terrace, about 9 mi wide, of Rhine Valley. Rhine R. at normal flow is at el 116; low water, 109; and high water, 130. It has avg slope of nearly 1'/mi. Cologne receives water from wells in gravels (about 100' thick) of valley fill. Starting in 1898 there have been about 200 observation wells installed around water works and throughout whole city. Water el and temps. in wells read regularly and wells also sampled for chem. and bact. tests, using special transportable pump for 2" wells. Ground water levels are subject to great changes, but are always above river el except near pumping station. Wells within 1,000' from Rhine show changes from 7' to 8'; wells 4 mi from river show changes of 4' and with delay of about 3 mo from changes in river level. Recharge from Rhine is small, and composition of ground water changes greatly from place to place. Cause of some changes was found to be waste dumps. It was calcd. that rainfall contributes about 24 mgd to ground water. Total pumpage is 36 mgd, and lowering of avg ground water level of 2"/yr accounts for only 3 mgd of deficit. Remainder of overpumpage must then come from Rhine or outside surface watershed. —*M. Suter*

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America help save water for this vital need . . . measure water accurately for homes and industry . . . provide a precise check of water usage and waste. And they make certain that users pay fairly for the water consumed . . . help make water departments self-supporting.

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Badger Water Meters

Badger Meter Mfg. Co.
Milwaukee 45, Wisconsin

"Measuring the water of the world for 50 years"

(Continued from page 52 P&R)

Leggette & Brashears, consulting ground water geologists, New York, announce the admission to partnership of Jack B. Graham. The firm name has been changed to Leggette, Brashears & Graham. From 1942 to 1954 Mr. Graham served in various capacities with the US Geological Survey.

George B. Bassett, founder of the Buffalo Meter Co., died Apr. 14, 1955, in his home in Buffalo, N.Y., at the age of 93. Born at Ballston Spa, N.Y., he received his early engineering training with the Watertown, N.Y., firm of Hinds and Moffett and the Sheffield School at Yale. At 25, he moved to Buffalo, where he and his brother, Edward, established an engineering and contracting business. His work on water system design led him to experiment with methods of improving existing water meters, and in 1892 he formed the Buffalo Meter Co. In the years since then, the company has grown to a point where its water meters are used in more than 2,500 communities; it also produces meters for other materials. Mr. Bassett retired from the presidency of the firm in 1945.

His business interests did not prevent him from devoting a great deal of his time to widespread civic activities. He was a director of such organizations as the Children's Aid Society, the YMCA, and the City Planning Assn. He conceived, but never claimed credit for, the idea of the Evershed Tunnel, whose construction made possible the first large-scale development of Niagara Falls water power. A Life Member of AWWA, he joined in 1909.

(Continued on page 88 P&R)

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Water Treatment

Efficient coagulation of surface or well waters. Aids taste and odor control. Effective in lime soda-ash softening. Adaptable to treatment of nearly all industrial waters.

Sewage Treatment

Efficient coagulation over wide range of pH values encountered in sewage treatment. Especially adapted for conditioning of sludge prior to vacuum or drying on sand beds.

SULFUR-DIOXIDE HIGHEST QUALITY SO_2

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SULFUR-DIOXIDE is effectively used for dechlorination in water treatment and to remove objectionable odors remaining after purification.

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COPPER SULFATE will control about 90% of the microorganisms normally encountered in water treatment plants more economically than any other chemical.

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Send postal card.

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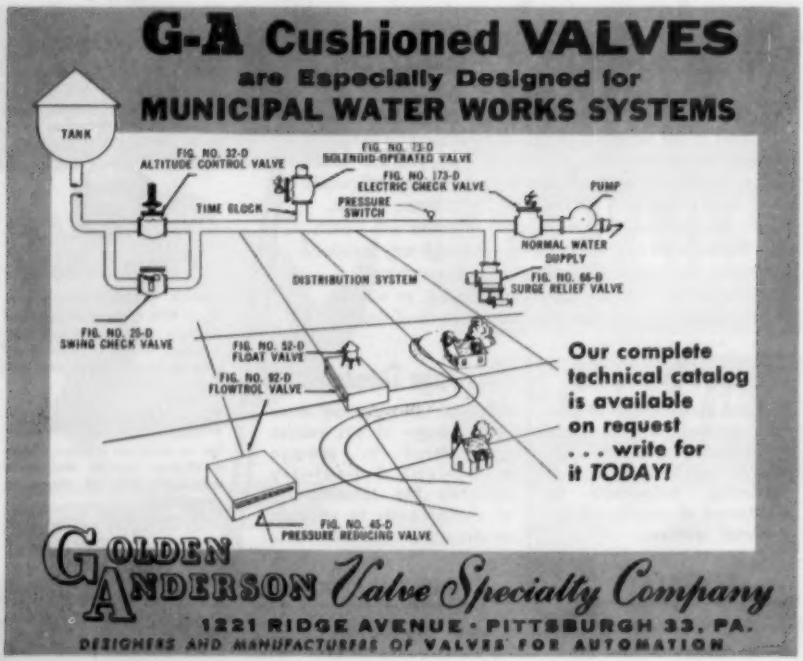
(*Continued from page 86 P&R*)

Safety in numbers, these days, is being reported by water works all over the country. From Richmond, Va., for instance, comes word of a filtration plant record of 377 days and 39,610 man-hours without an injury. From Chicago's Water Purification Div. we learn that an average of 218 employees working a total of 422,315 man-hours in 1954 sustained only 21 nondisabling injuries, and not a single disabling injury resulted in loss of pay. And from the East Bay Municipal Utility Dist., Oakland, Calif., which passed its 1,000,000-man-hour no-injury mark last September, we note a 1954 record of only 15 lost-time injuries in more than 2,300,000 man-hours of work.

It is the size of these numbers in safety, perhaps, that discourages the average water worker from getting involved. But the fact that most water works have to work water for 100 years or more to achieve 1,000,000 man-hours, with or without injury, is hardly an argument against safety—or even numbers. After all, the one-man water works that manages 6 months without a disabling injury will relatively have outdone East Bay's 1,000,000-man-hour record. And a good thing, too, with 100 per cent of the staff in jeopardy. All of which would seem to suggest that, in safety, the smaller the numbers, the more they count. "Not so, though," say those who know, "safety is a numbers game at which everybody wins!"

The Harry E. Jordan Scholarship Committee has announced its first award, for 1955–56. The recipient is Paul A. Kuhn, of Chicago, a graduate of the University of Illinois, Cham-

(*Continued on page 90 P&R*)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MUNICIPAL WATER WORKS SYSTEMS
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FIG. NO. 73-D SOLENOID-OPERATED VALVE
FIG. NO. 173-D ELECTRIC CHECK VALVE
PUMP
TIME CLOCK
PRESSURE SWITCH
NORMAL WATER SUPPLY
DISTRIBUTION SYSTEM
FIG. NO. 66-D SURGE RELIEF VALVE
FIG. NO. 25-D SWING CHECK VALVE
FIG. NO. 52-D FLOAT VALVE
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Nashville goes to 100 mgd with

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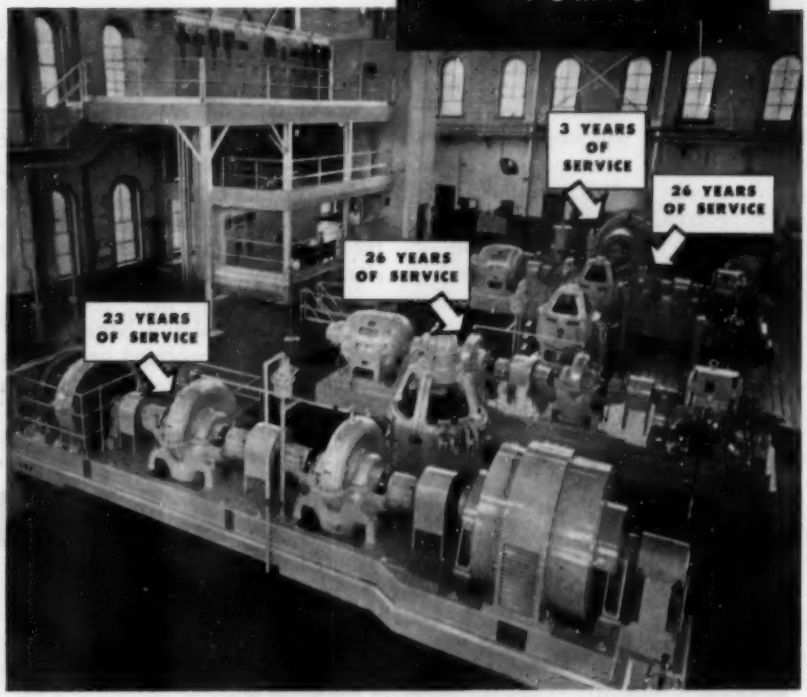
CENTRIFUGAL PUMPS



The years of service shown on these De Laval centrifugal pumps tell their own story of dependability and progress. By revising its facilities, the George Reyer pumping station in Nashville has boosted its pumping capacity to 100 million gallons per day.

The De Laval pumps, which have been in service for years, play an integral part in this more efficient operation. The newest De Laval unit handles 40 million gallons per day.

In fact 80% of all cities in the United States, with a population of 100,000 or over, use De Laval centrifugal pumps. Capacities of De Laval centrifugal water works pumps range from 100 thousand to 100 million gallons per day. Write for your copy of new De Laval Bulletin 1004.

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DL 5084

(Continued from page 88 P&R)

paign (B.S. in Sanitary Engineering, 1954). The $1,500 award, sponsored by the Water and Sewage Works Manufacturers Assn., is intended to further the postgraduate studies of a deserving resident of the area in which the site of the Annual Convention is located. Mr. Kuhn, who has been employed by the Waukegan, Ill., water department and presently works for the Pacific Flush Tank Co., will now be able to devote full time to studying for a master's degree at Northwestern University Technological Institute, Evanston, Ill. The formal presentation of the award is to be made on the evening of Jun. 13 at the Chicago Convention.

J. E. Kunkler has been named sales manager of the Southwestern Div. of Armco Drainage & Metal Products. J. B. Evans will succeed him as manager of Southern California and Arizona sales for the Calco Div. of Armco Drainage.

An improved variable-speed laboratory shaker that will operate continuously, without fluctuation, at any rate between 60 and 260 oscillations per minute is offered by Eberbach Corp., Ann Arbor, Mich. A $\frac{1}{3}$-hp split-phase induction motor, operating at fixed speed and maximum torque, actuates the shaker platform through a mechanical speed control.

Carl L. Nickel and Norman C. Olson have been appointed product sales managers—for water systems and

(Continued on page 92 P&R)

Depend on Cyanamid's ALUM for 6 good reasons

1. It feeds uniformly, without trouble, in solid or liquid form.
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4. It gives maximum adsorption of suspended and colloidal impurities.
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6. It is available in granular form or in the new easy-to-use liquid form.

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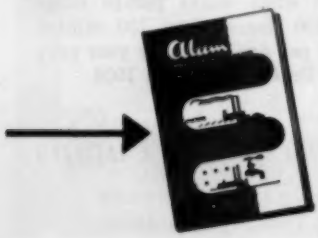

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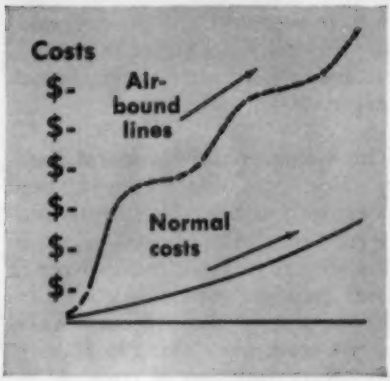
HOW TO CUT PUMPING COSTS

• When air lodges in high spots, it cuts the effective area of pipe, creates a friction head, lowers pumping capacity . . . and may cause water-hammer.

Simplex Air Release Valves end these troubles. They vent air automatically to eliminate binding, increase efficiency, cut pumping costs. Simple, rugged 2" size, for up to 250 psi., costs you little, quickly pays for itself! BULLETIN 1204



HOW TO PREVENT PIPELINE COLLAPSE

• A break in the line or a quickly opened valve often releases water at terrific velocity. Internal pressure falls below atmospheric. Thin-walled pipelines collapse!

Simplex Air Inlet Valves quickly admit air, prevent a vacuum, protect lines from collapse. They also vent air when lines are being filled to stop binding, increase efficiency. Standard 4" to 10" sizes.
BULLETIN 1202

Pipeline collapse due to vacuum

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Accurate instruments and controls since 1904

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VALVE AND METER COMPANY

(*Continued from page 90 P&R*)

dealer line products and for engineered line products, respectively—by Peerless Pump Div., Food Machinery & Chemical Corp., Los Angeles. Both men have been with the division since 1946, Nickel as a product sales manager and Olson as Chicago branch manager.

The Conference of Federal Sanitary Engineers has recently been formed, with membership open to sanitary engineers on active duty as armed forces officers or civilian employees of federal agencies. Conference aims include support of the proposed Intersociety Certification Board in Sanitary Engineering; solution of training, recruitment, and career-planning problems; and cooperation with existing technical and professional societies. Information and membership applications may be obtained from the secretary of the organization, Lt. Col. Alvin Meyer, 1701 South 78th St., Omaha 6, Neb.

An extension lubricator for injecting lubricant as packing in pump shaft stuffing boxes is now being offered as a stock item by the Meter & Valve Div. of Rockwell Mfg. Co., Pittsburgh. The accessory is available in brass or cadmium-plated steel.

Henry D. Hughes has been placed in charge of the Dallas sales office of Reilly Tar & Chemical Corp. Mr. Hughes has been engaged in the water and sewage construction business.

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The spun lining of coal tar enamel provides the smoothest interior surface, which means a maximum flow of water and lowest pumping costs.

You no longer have to specify over-sized pipe to compensate for future corrosion "shrinkage". Protective coating being corrosion proof enables you to save on steel as well.

To wrap up those savings—specify steel pipe—lined and coated with coal tar enamel, of course.

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Applications received Apr. 1–30, 1955

Ace Pipe Cleaning, Inc., Wallace A. Marsh, Pres., 3513 E. 18th St., Kansas City, Mo. (Assoc. M. Apr. '55)

Albertsen, Hans C., Engr., General Filter Co., 4345 Madison Ave., Kansas City, Mo. (Apr. '55) *P*

Arceivala, S. J., Engr., Bombay Munic. Corp., Water Dept., Improvement Trust Bldg., Napier Rd., Fort, Bombay 1, India (Apr. '55) *MRPD*

Arizona Water Co., R. E. Polenske, Asst. Gen. Mgr., Box 5347, Phoenix, Ariz. (Corp. M. Apr. '55) *MRP*

Barney, Casper; *see* Eau Claire (Wis.) Munic. Water Utility.

Barton, Owen Charles, Chief Chemist, Island Filtration Plant, Centre Island, Toronto, Ont. (Apr. '55) *P*

Beck, John D., Supt., Filtration Div., Dept. of Water, City-County Bldg., Pittsburgh, Pa. (Apr. '55) *MP*

Belsom, Richard A., Mfer.'s Repr., Frieshon & Belsom, Box 7044, New Orleans, La. (Apr. '55)

Blair, Alvin, Water Works Contractor, 244 Paul Rd., Rochester, N.Y. (Apr. '55) *D*

Bolleau, Clifton W., Prin. Mech. Engr., Design Div., Tennessee Valley Authority, Arnstein Bldg., Knoxville, Tenn. (Apr. '55) *P*

Bourgeois, Roy J., Field Supt., Lafourche Parish Water Dist. No. 1, Lockport, La. (Apr. '55) *M*

Brackin, Horace W., Supt., Water Works, Marion, Ark. (Apr. '55)

Branum, Jack S.; *see* Hoisington (Kan.)

Brennan, William, Supt., Water Dept., Maple Ave., Ashburnham, Mass. (Apr. '55) *MD*

Brinkman, Charles E., Chief Accountant, Erie County Water Authority, 338 Ellicott Square, Buffalo 3, N.Y. (Apr. '55) *M*

Brown, Ralph H.; *see* Rio Plaza Water Co.

Burnett, John J., 1917—22nd St., Niagara Falls, N.Y. (Apr. '55)

Bush, Julius Moore, Dist. Mgr., Fischer & Porter Co., 1312 Broadway, Denver, Colo. (Apr. '55) *RP*

Campbell, H. Ross, Supervisor, Building Crafts, Creole Pet Corp., Cabimas, Est. Zulia, Venezuela (Apr. '55) *P*

Campbell, Oran G., Draftsman, Water Dept., Distr. System, 2121 Westwood Blvd., Oklahoma City, Okla. (Apr. '55) *D*

Chandler, R. L., Engr., Kansas City Suburban Water Co., 5916 Dearborn, Mission, Kan. (Apr. '55) *PD*

Chapman, Cecil Carey, Partner, Chapman Eng. Co., Box 401, Williston, S.C. (Apr. '55) *MRPD*

Coffey, H. Douglas, Pipe Sales, Lynchburg Foundry Co., 122 S. Michigan Ave., Chicago, Ill. (Apr. '55)

Corey, Quentin M., Supt., City Hall, West Linn, Ore. (Apr. '55) *M*

Croll, Jesse D., Sales Engr., Graver Water Conditioning Co., 216 W. 14th St., New York 11, N.Y. (Apr. '55) *MP*

Davis, C. Lamar, Research Engr., Black & Assocs., Inc., 700 S.E. 3rd St., Gainesville, Fla. (Apr. '55) *PD*

DeArmond, Gene V., Civ. Engr., Dean S. Kingman, 153 University Ave., Palo Alto, Calif. (Apr. '55) *P*

DeWire, Harry A., Regional San. Engr., State Dept. of Health, Box 90, Harrisburg, Pa. (Apr. '55) *P*

DeZurik, David; *see* DeZurik Shower Co.

DeZurik Shower Co., David DeZurik, Pres., Sartell, Minn. (Assoc. M. Apr. '55)

Duttamel, Notley Young, Jr., Salesman, Centriline Corp., 1701 Union Bank Bldg., Pittsburgh 22, Pa. (Apr. '55) *D*

Eau Claire Munic. Water Utility, Casper Barney, Supt., City Hall, Eau Claire, Wis. (Munic. Sv. Sub. Apr. '55) *MP*

Ericson, Milton O., Salesman, General Chemical Div., 3357 W. 47th Pl., Chicago 32, Ill. (Apr. '55) *P*

Evans, George R., Vice-Pres. & Gen. Mgr., Glenfield & Kennedy Inc., 677—5th Ave., New York 22, N.Y. (Apr. '55) *P*

Ferreira, Charles Joseph, Supt. of Streets & Water, Pittsburg, Calif. (Apr. '55) *M*

Foster, John B., City Engr., Norton, Kan. (Apr. '55) *MD*

Frizell, Robert E., Supt., Water Dept., 3rd St. & Simpson Ave., National Park, N.J. (Apr. '55) *MRPD*

Giauque, David H., Asst. Mill Supt., Asbestos Corp. Ltd., King Mine, Thetford Mines, Que. (Apr. '55) *MD*

Glass, Harry J., II, Gen. Partner, Harry J. Glass & Assocs., Box 1085, Denver 1, Colo. (Apr. '55)

Glazener, D. C., Water & Sewer Supt., 1701 Wood Ave., Killeen, Tex. (Apr. '55) *R*

Gowen, Howerton; *see* Gowen, Howerton, Co.

Gowen, Howerton, Co., Inc., Howerton Gowen, Pres., E. 11th St., Roanoke Rapids, N.C. (Assoc. M. Apr. '55)

Grundeman, E., Gen. Contractor, 1427 W. 37th St., Topeka, Kan. (Apr. '55) *D*

Hamilton, Don H.; *see* Kingman (Kan.)

Hamilton, Roy A., Civ. Eng. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles, Calif. (Apr. '55) *R*

Hanratty, James J., Dist. Mgr., Graver Water Conditioning Co., 5343 McCormick, Box 1764, Houston, Tex. (Apr. '55) *P*

Harris, E. B., City Secy., Clifton, Tex. (Apr. '55) *M*

Hayes, Aubrey, Supt. of Utilities, Municipal Bldg., Salem, Ohio (Apr. '55) *MP*

Hayes, Walter D., Asst. Engr., Warren & Van Praag, Inc., 253 S. Park St., Decatur, Ill. (Apr. '55) *P*

Healy, D. L.; *see* Healy-Ruff Co.

Healy-Ruff Co., D. L. Healy, Pres., 2255 University Ave., St. Paul 14, Minn. (Assoc. M. Apr. '55)

Heikkila, Russell E., Water Supt., Box 604, Cosmopolis, Wash. (Apr. '55) *D*

Hoisington, City of, Jack S. Branum, City Mgr., Municipal Bldg., Box 28, Hoisington, Kan. (Corp. M. Apr. '55) *MRPD*

Holbert, Leon R., Field Instructor, Texas Eng. Extension Service, Box 236 F.E., College Station, Tex. (Apr. '55) *MP*

Horstman, Herman G., Vice-Pres., Indiana Gas & Water Co., Inc., 1630 N. Meridian St., Indianapolis 2, Ind. (Apr. '55) *M*

Houser, Donald F., Civ. Engr., Casler & Stapleton, 15 Havendale Court, Jacksonville, Ill. (Apr. '55) *RP*

Hursey, Francis J.; *see* Pass Christian (Miss.)

Illig, George L., Jr., Asst. Mgr., Threshold Treatment Div., Calgon, Inc., 323—4th Ave., Pittsburgh 22, Pa. (Apr. '55) *RP*

Imhof, Leonard J., Civ. Engr., F. E. Devlin, 4005 E. Kellogg, Wichita 17, Kan. (Apr. '55) *RPD*

Jeffrey, Edgar A., Graduate Student & Research Asst., 918 Iowa Ave., Iowa City, Iowa (Jr. M. Apr. '55) *MRP*

Johnson, Beverly M., Civ. Engr., Whitman, Requardt & Assoc., 2 W. Preston St., Baltimore, Md. (Apr. '55) *RPD*

Kingman, City of, Don H. Hamilton, Supt. of Utilities, Kingman, Kan. (Corp. M. Apr. '55) *MRPD*

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Leslie, John M., Supt. & Chemist, Water & Sewage Plants, 103 Mitchell Ave., Salisbury, N.C. (Apr. '55)

Levander, C. G., Cons. Engr., Seaholm & Levander, 815 Airport Blvd., Austin, Tex. (Apr. '55)

Lobdell, Wentworth N., Pres., Edgebrook Supply Co. & North Park Water Supply Co., 402 Rockford Trust Bldg., Rockford, Ill. (Apr. '55) *M*

Lord, G. Ross, Prof., Mech. Eng., Univ. of Toronto, Toronto 5, Ont. (Apr. '55)

Lowe, George E., Walton-Vikings Co., 5123 Buena Vista, Kansas City, Kan. (Apr. '55) *D*

Lowry, Homer E., City Supt., Winnsboro, La. (Apr. '55) *P*

Lyle, Jack C., Div. Mgr., The Ludlow Valve Mfg. Co., Inc., 609 Glenn Bldg., Atlanta, Ga. (Apr. '55)

Marsh, Wallace A.; *see* Ace Pipe Cleaning

Mathews, Dean S., Engr., State Dept. of Health, Bureau of San. Eng., Box 1741, Fort Worth, Tex. (Apr. '55) *RP*

Matlock, Luther J., Operating Engr., Water & Sewage, Perrin Air Force Base, Sherman, Tex. (Apr. '55) *MRP*

Michail, Casimer, Salesman, Hays Mfg. Co., 801 W. 12th St., Erie, Pa. (Apr. '55) *D*

Miller, C. B., Supt., Filtration Plant, 114 N. Lamar St., Eastland, Tex. (Apr. '55) *P*

Minneapolis Water Dept., Rollind H. Olson, Supt., Minneapolis, Kan. (Corp. M. Apr. '55) *MRPD*

Moeller, Leroy E., Sr. Asst. Engr., Water Dept., 201 W. Main St., Alhambra, Calif. (Apr. '55) *MRD*

Morrill, Leigh W., Engr., Water Dept., Essex Fells, N.J (Apr. '55) *MRPD*

Mortensen, A. P.; *see* Myrtle Creek (Ore.)

Muller, Stephen, Sales Repr., General Chemical Div., 3357 W. 47th Pl., Chicago, Ill. (Apr. '55) *P*

Myers, Cyril B., Administrative Vice-Pres. & Treas., New York Water Service Corp., 132 W. 43rd St., New York 36, N.Y. (Apr. '55) *M*

Myrtle Creek, City of, A. P. Mortensen, City Supt., 206 Pleasant St., Myrtle Creek, Ore. (Munic. Sv. Sub. Apr. '55) *MPD*

Nasi, Kaarlo W., Director, US Public Health Service, Water Supply & Pollution Control Program, 6009 Dellwood Pl., Bethesda, Md. (Apr. '55)

Nathan, Walter S., Owner, Water & Waste Conditioning Co., 3826 N. Bartlett Ave., Milwaukee 11, Wis. (Apr. '55) *P*

Neumann, Harry G., Research Microbiologist, Dept. of Water & Power, Box 3669 Terminal Annex, Los Angeles 54, Calif. (Apr. '55)

New Chicago Water Works, Carl M. Raezka, Supt., 3612 Michigan Ave., Hobart, Ind.. (Corp. M. Apr. '55) *MRPD*

Nolte, Frank A., Mgr., Pump Dept., Fairbanks, Morse & Co., 9th & Harney Sts., Omaha, Neb. (Apr. '55) *D*

Nonaes, Waldemar B., Foreman, Louisville Asphalt Co., 240 E. Lee St., Louisville 8, Ky. (Apr. '55) *MPD*

Norris, Willard N., City Mgr., City of Springfield, 293 Ave. C, P.O. Box Battle Creek Mich. (Apr. '55) *MPD*

Olson, Rollind; *see* Minneapolis (Kan.) Water Dept.

Orland, Herbert P., Assoc. Editor, Scranton Publishing Co., 708 S. Prospect, Champaign, Ill. (Apr. '55) *MRPD*

Pass Christian, City of, Francis J. Hursey, Mayor, Pass Christian, Miss. (Corp. M. Apr. '55)

Payton, Louis S., Public Health Engr., Missouri Div. of Health Dist. No. 3, Sedalia, Mo. (Apr. '55) *P*

Pfeil, T. D., Owner & Operator, Water Plant, Kirby, Tex., Rte. 1 Box 53, Converse, Tex.

Polenske, R. E.; *see* Arizona Water Co.

Preiss, LeRoy; *see* Stafford (Kan.)

Raezka, Carl M.; *see* New Chicago Water Works

Rio Plaza Water Co., Inc., Ralph H. Brown, Mgr., Box 287, Oxnard, Calif. (Corp. M. Apr. '55) *MR*

Ritchie, G. Wells, Editor, *Civic Administration,* 481 University Ave., Toronto 2, Ont. (Jan. '55)

Roberts, Charles Lee, Civ. Eng. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles 12, Calif. (Apr. '55) *MRD*

Roberts, John B., Student, Univ. of Florida, 1239—10th Ave., N.W., Gainesville, Fla. (Jr. M. Apr. '55) *RP*

Ruby, John W., Asst. Supt., Water Dept., 919 Sunset Rd., Ann Arbor, Mich. (Apr. '55) *P*

Ruland, Willis E., Civ. Engr., Corps of Engrs., US Army, 1709 Jackson, Omaha, Neb. (Apr. '55) *MRPD*

Rutter, Alvin Isaac, Supt., Filtration Plant, Utilities Board, Bristol, Va. (Apr. '55) *P*

Sanborn, K. F., City Mgr., Wamego, Kan. (Apr. '55) *MRPD*

Schleck, John C., Hydr. Engr., Gibbs & Hill Inc., Pennsylvania Stn., New York 1, N.Y. (Apr. '55) *D*

Schulz, Robert, Sales Engr., T. S. McShane Inc., 1115 Howard, Omaha, Neb. (Apr. '55) *PD*

Self, Edward C., Asst. Supervisor, Power Operations, Kaiser Aluminum & Chemical Corp., Box 1600, Chalmette, La. (Apr. '55) *MP*

Smith, Horace L., City Engr., City Hall, Box 1156, Snyder, Tex. (Apr. '55) *MPD*

Stafford, City of, LeRoy Preiss, Supt. of Utilities, Stafford, Kan. (Corp. M. Apr. '55) *MRD*

Stansberry, Ralph L., Asst. Supt., Water Dept., Holden, Mo. (Apr. '55) *MPD*

Swain, Charles H., Sr. Civ. Engr., Water Dept., 1210—5th St., N.E., Minneapolis 13, Minn. (Apr. '55) *MP*

Syme, Ian, Sales Mgr., Vaughan & Shaw Co., Ltd., 921 Dominion Square Bldg., Montreal, Que. (Jan. '55)

Tejada Andrade, Rene, Civ. Engr., Water Supply Dept., Calle Murillo No. 292, La Paz, Bolivia (Apr. '55) *MRPD*

Thomas, Feriys W., Secy. & Gen. Mgr., Carpinteria County Water Dist., Box 578, Carpinteria, Calif. (Apr. '55) *M*

Turek, James L., Sales Engr., De Laval Steam Turbine Co., 122 S. Michigan Ave., Chicago 3, Ill. (Apr. '55) *M*

Udstuen, Arnold J., Supervisor, Meter Repair, Water Dept., 29th & Cambria, Philadelphia, Pa. (Apr. '55) *M*

Usher, T. Ivan, Personnel & Safety Supervisor, Dept. of Public Utilities, Water Div., Box 1639, Tacoma, Wash. (Apr. '55) *M*

Van Natta, Ronald C., Operator, Water Dept., 661 Illinois Ave., Elgin, Ill. (Apr. '55) *MP*

Wager, Aldwyn, City Mgr., Harbor Springs, Mich. (Apr. '55) *MD*

Wakefield, John W., 3690 Oak St., Jacksonville 5, Fla. (Apr. '55)

Ware, Judson; *see* Lake Placid (N.Y.)

Washington, Donald R., Graduate Student, San. Eng., Univ. of California, 2220 Carleton St., Berkeley 4, Calif. (Jr. M. Apr. '55)

Wedge, H. W., City Mgr., Clare, Mich. (Apr. '55) *MD*

Welch, John W., Jr., Mgr., Brinkley Water Co., Inc., Box 432, Brinkley, Ark. (Apr. '55) *M*

Williams, Richard L., Sales Repr., Sebastian Diesel Equipment Co., 1801 Joplin St., Joplin, Mo. (Apr. '55)

Wilt, Marlin E., Dist. San. Engr., State Dept. of Health, 138 E. 3rd St., Williamsport, Pa. (Apr. '55) *MRPD*

Wittenberg, Sol J., Comr., Div. of Water, 565 N. Erie St., Toledo 2, Ohio (Apr. '55) *MRPD*

Yenchko, John, San. Engr., Kappe & Assocs., 5200 Wisconsin Ave., N.W., Washington 15, D.C. (Apr. '55) *P*

Yonkers Dept. of Public Health, Arthur Wallach, Director, Div. of Food & Sanitation, Health Center Bldg., Yonkers, N.Y. (Munic. Sv. Sub. Apr. '55)

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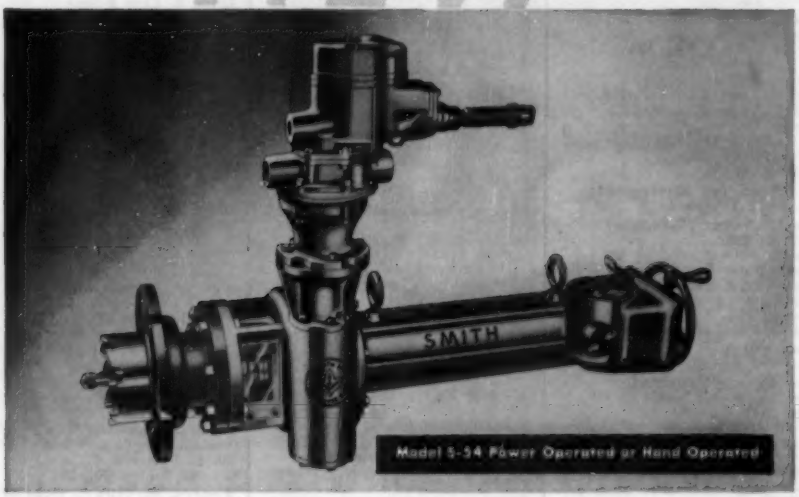

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Badger Meter Mfg. Co.
Buffalo Meter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Builders-Providence, Inc.
Fischer & Porter Co.
Foster Eng. Co.
Infilco Inc.
Simplex Valve & Meter Co.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Builders-Providence, Inc.
Fischer & Porter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Simplex Valve & Meter Co.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Mixing Equipment:
Chain Belt Co.
General Filter Co.
Infilco Inc.

Ozonation Equipment:
Welsbach Corp., Ozone Processes Div.

Paints
Inertol Co., Inc.

Pipe, Asbestos-Cement:
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
American Brass Co.

Pipe, Cast Iron (and Fittings):
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Concrete Pressure Pipe Assn.
American Pipe & Construction Co.
Lock Joint Pipe Co.

Pipe, Copper:
American Brass Co.

Pipe, Steel:
Alco Products, Inc.
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.

Pipe Coatings and Linings:
The Barrett Div.
Cast Iron Pipe Research Assn.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Reilly Tar & Chemical Corp.

Pipe Cutters
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Jos. G. Pollard Co., Inc.

Reed Mfg. Co.
A. P. Smith Mfg. Co.
Spring Load Mfg. Corp.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pipe Vises
Spring Load Mfg. Corp.

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Foster Eng. Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Morse Bros. Mchy. Co.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
Infilco Inc.
Proportioneers, Inc.
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Morse Bros. Mchy. Co.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Layne & Bowler, Inc.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Fischer & Porter Co.
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Fischer & Porter Co.
Infilco Inc.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Chicago Bridge & Iron Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

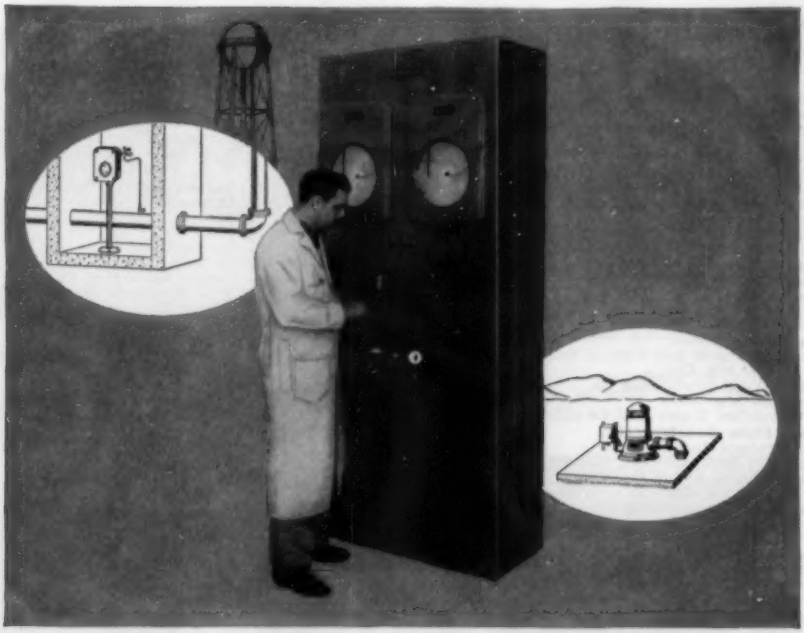
Sand Expansion Gages; see Gages

Sleeves; see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
General Filter Co.
Graver Water Conditioning Co.
Permutit Co.

Sodium Hexametaphosphate:
Blockson Chemical Co.
Calgon, Inc.



SYSTEM SUPERVISION IS SIMPLE AND ECONOMICAL WITH *Synchro-Scan**

ANOTHER B-I-F Blue Chip PRODUCT

Builders-Providence Synchro-Scan Supervisory Control gives you the economy of being able to operate remote equipment from a centralized control station — plus the extra economy of extreme simplicity: — basically right in its design, easy to operate, and easy to maintain.

May we tell you about Synchro-Scan Supervisory Control — the only control with the unique "building block" design. Send, today, for new Bulletin 240-M6. Builders-Providence, Inc., 365 Harris Avenue, Providence 1, Rhode Island.

*Patent applied for

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DIVISION OF B-I-F INDUSTRIES, INC.

BUILDERS IRON FOUNDRY • PROPORTIONERS, INC. • OMEGA MACHINE CO.

B/F METERS FEEDERS CONTROLS

Sodium Silicate
Philadelphia Quartz Co.
Softeners:
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Softening Chemicals and Compounds:
Calgon, Inc.
Cochrane Corp.
General Filter Co.
Infilco Inc.
Morton Salt Co.
Permutit Co.
Tennessee Corp.
Standpipes, Steel:
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.
Steel Plate Construction:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.
Stops, Curb and Corporation:
Hays Mfg. Co.
Mueller Co.
Welsbach Corp., Kitson Valve Div.
Storage Tanks; see Tanks
Strainers, Suction:
James B. Clow & Sons
M. Greenberg's Sons
Johnson, Edward E., Inc.
R. D. Wood Co.
Surface Wash Equipment:
Cochrane Corp.
Permutit Co.
Swimming Pool Sterilization:
Everson Mfg. Corp.
Fischer & Porter Co.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc.
Wallace & Tiernan Inc.
Welsbach Corp., Ozone Processes Div.
Tanks, Steel:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Water Conditioning Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.
Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.
Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.
Welsbach Corp., Kitson Valve Div.
Taste and Odor Removal:
Cochrane Corp.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Industrial Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc.
Wallace & Tiernan Inc.
Welsbach Corp., Ozone Processes Div.
Tenoning Tools
Spring Load Mfg. Corp.
Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.
Turbines, Steam:
DeLaval Steam Turbine Co.
Turbines, Water:
DeLaval Steam Turbine Co.
Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Kensselaer Valve Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.
Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.
Valves, Altitude:
Ross Valve Mfg. Co., Inc.
Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Shower Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Kensselaer Valve Co.
R. D. Wood Co.
Valves, Detector Check:
Hersey Mfg. Co.
Valves, Electrically Operated:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
Valves, Float:
James B. Clow & Sons
Henry Pratt Co.
Ross Valve Mfg. Co., Inc.
Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.
Valves, Hydraulically Operated:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
DeZurik Shower Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.
Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.
Valves, Regulating:
DeZurik Shower Co.
Foster Eng. Co.
Mueller Co.
Henry Pratt Co.
Ross Valve Mfg. Co.
Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.
Waterproofing
Inertol Co., Inc.
Water Softening Plants; see Softeners
Water Supply Contractors:
Layne & Bowler, Inc.
Water Testing Apparatus:
Wallace & Tiernan Inc.
Water Treatment Plants:
Allis-Chalmers Mfg. Co.
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr-Oliver Inc.
Fischer & Porter Co.
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.
Welsbach Corp., Ozone Processes Div.
Well Drilling Contractors:
Layne & Bowler, Inc.
Well Screens
Johnson, Edward E., Inc.
Wrenches, Ratchet:
Dresser Mfg. Div.
Zeolite; see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1953 AWWA Directory.

Chosen in the West. "Century" Pipe is shown here as it is installed at Lake Almanor, California. The piping will be a part of the water system at Lake Almanor Country Club Estates.

Chosen in the East. This photograph shows "Century" Pipe being laid in the hard, rock-filled soil of New England near South Hadley Falls, Mass. Today, more and more value-conscious communities are specifying this durable, long-lasting pipe.

13 miles of long-lasting pipe! Pictured here is part of a 13-mile installation of "Century" Pipe at the Lakewood Water District near Tacoma, Washington. Made of two practically indestructible materials, asbestos and cement, "Century" Pipe assures *extra* years of low-cost service.

Across America, K&M "Century"® Pipe is Stabilizing Pumping Costs

Because K&M "Century" Asbestos-Cement Pipe is non-tuberculating, highly resistant to corrosion, and immune to electrolysis, the diameter of its bore never changes. Consequently, there is *no creeping up of pumping costs.*

Durable "Century" Pipe, formed on a smooth steel mandrel, has a smooth interior surface offering a minimum of friction. The Williams and Hazen Constant "C" is conservatively placed at 140. As it is relatively light in weight, machinery for handling up to and including 12" pipe sizes is not needed.

Leakage problems also vanish! Permanently tight joints are made possible by the use of "Century" Simplex Couplings. The pipe also meets the A.W.W.A., A.S.T.M., and Federal specifications for asbestos-cement pressure pipe.

Write for FREE booklet, "Mains Without Maintenance." We'll also be glad to help you with any installation problem in your specific operation.

See us at Booths No. 11 and 12, American Water Works Association Convention in Chicago, June 12-17.

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Little Things Like Stainless Steel Trim For The Best Corrosion Protection

Many potable waters are corrosive to meters—some more so than others. Rockwell guards its Arctic and Tropic meters against corrosive attack through the use of carefully selected and treated stainless steel at such vulnerable points as the division plate, driving dog, disc spindle, stuffing box spindle and screws.

If corrosion is a problem with you, contact your nearby Rockwell office for the best advice and protection you can get.

The Symbol for Service, Quality and Performance in Water Meters

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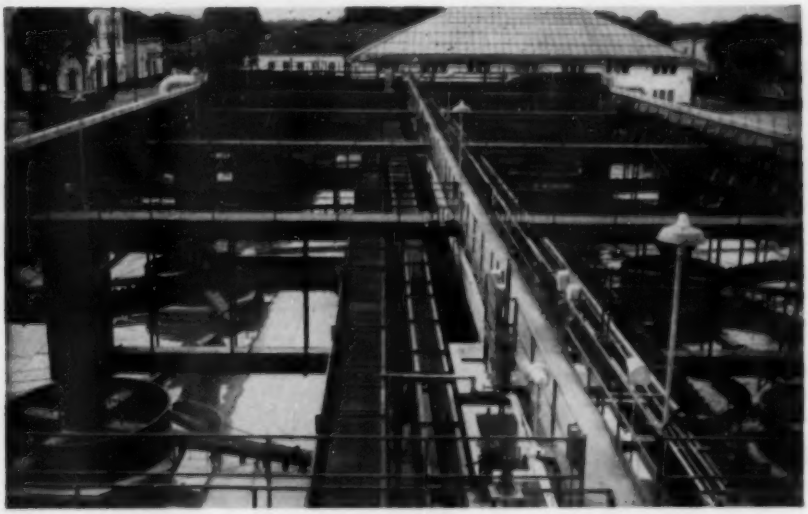
Los Angeles Midland, Texas N. Kansas City, Mo. New York Philadelphia Pittsburgh
San Francisco Seattle Shreveport Tulsa

In water treatment problems...

you won't find identical twins

No two water treatment problems are exactly alike. The right solution to each one only be arrived at after a careful study of the local conditions. Variables such as raw water composition, rate of flow and results required automatically rule out the cure-all approach. The installation shown below is a good example of how equipment should be selected to fit the job . . . and not vice versa.



New Orleans, Louisiana | *Gets Immediate Softening and Clarification Capacity While Long Range Improvements are Made*

How to get immediate high-rate softening and clarification capacity for 100 MGD while long range improvements were made in another section of the plant . . . that was the problem faced by New Orleans.

A study of existing facilities showed that effective use could be made of two existing basins which had been used as grit chambers in the original plant. Twelve Dorrco Hydro-Treator* mechanisms, each 54′ 6″ diameter, were installed . . . six in each basin, with the basins operating in parallel at 50 MGD capacity each.

Other improvements now being completed in the plant include the addition of two Dorrco Monorake* units in one of the existing coagulation basins.

If you'd like more information on the complete line of Dorr-Oliver water treatment equipment, write for Bulletin No. 9141, Dorr-Oliver Incorporated, Stamford, Conn., or in Canada, 26 St. Clair Avenue E., Toronto 5.

Every day, nearly 8 billion gallons of water are treated with Dorr-Oliver equipment.

*Trademark Reg. U. S. Pat. Off.

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WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT
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Jointed for . . . Permanence with LEADITE

Generally speaking, most Water Mains are buried beneath the Earth's surface, to be forgotten,—they are to a large extent, laid for permanency. Not only must the pipe itself be dependable and long lived,—but the joints also must be tight, flexible, and long lived,—else leaky joints are apt to cause the great expense of digging up well-paved streets, beautiful parks and estates, etc.

Thus the "jointing material" used for bell and spigot Water Mains MUST BE GOOD,—MUST BE DEPENDABLE,—and that is just why so many Engineers, Water Works Men and Contractors aim to PLAY ABSOLUTELY SAFE, by specifying and using LEADITE.

Time has proven that LEADITE not only makes a tight durable joint,—but that it improves with age.

The pioneer self-caulking material for c. i. pipe.
Tested and used for over 40 years.
Saves at least 75%

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No Caulking